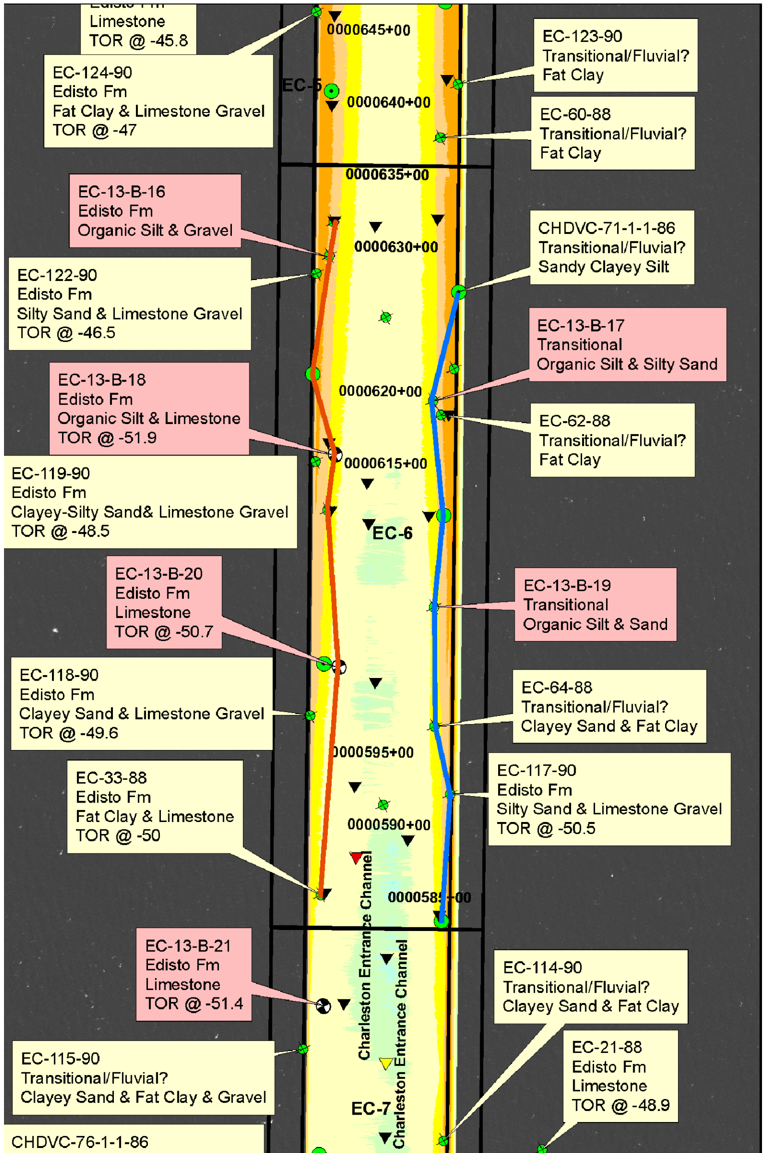
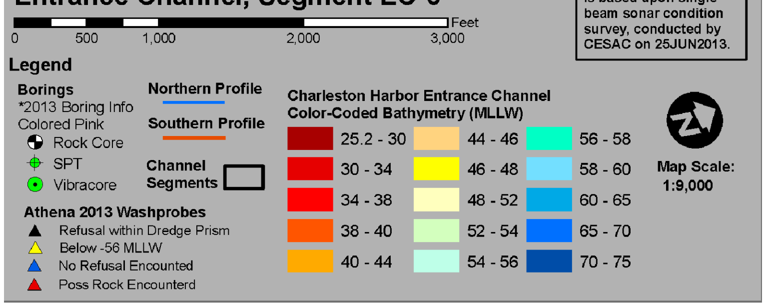


Figure B-55: Harbor Stratigraphy Entrance Channel, Segment EC-6

NOTE: Bathymetric color-contoured surface is based upon single-beam sonar condition survey, conducted by CESAC on 25JUN2013.



Map Scale:
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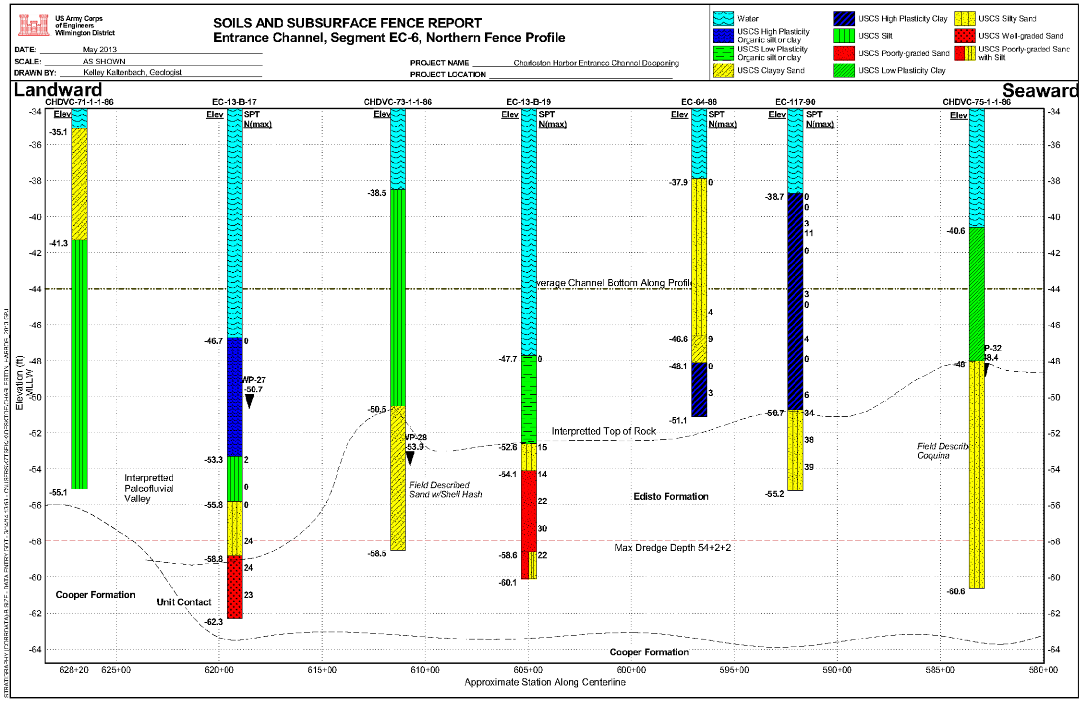
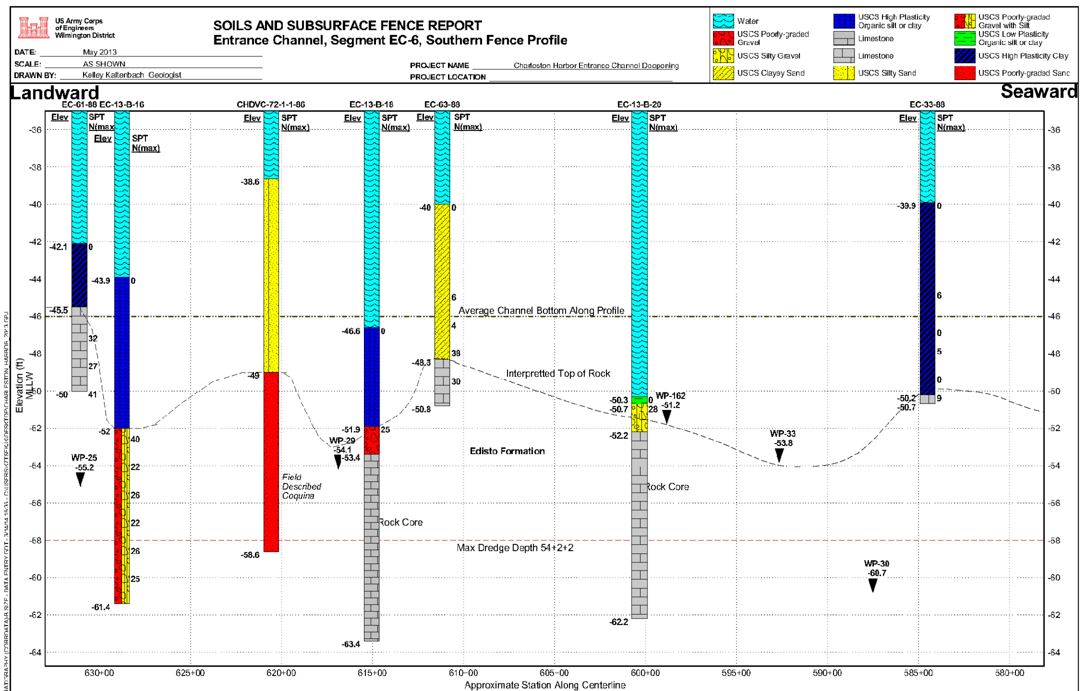




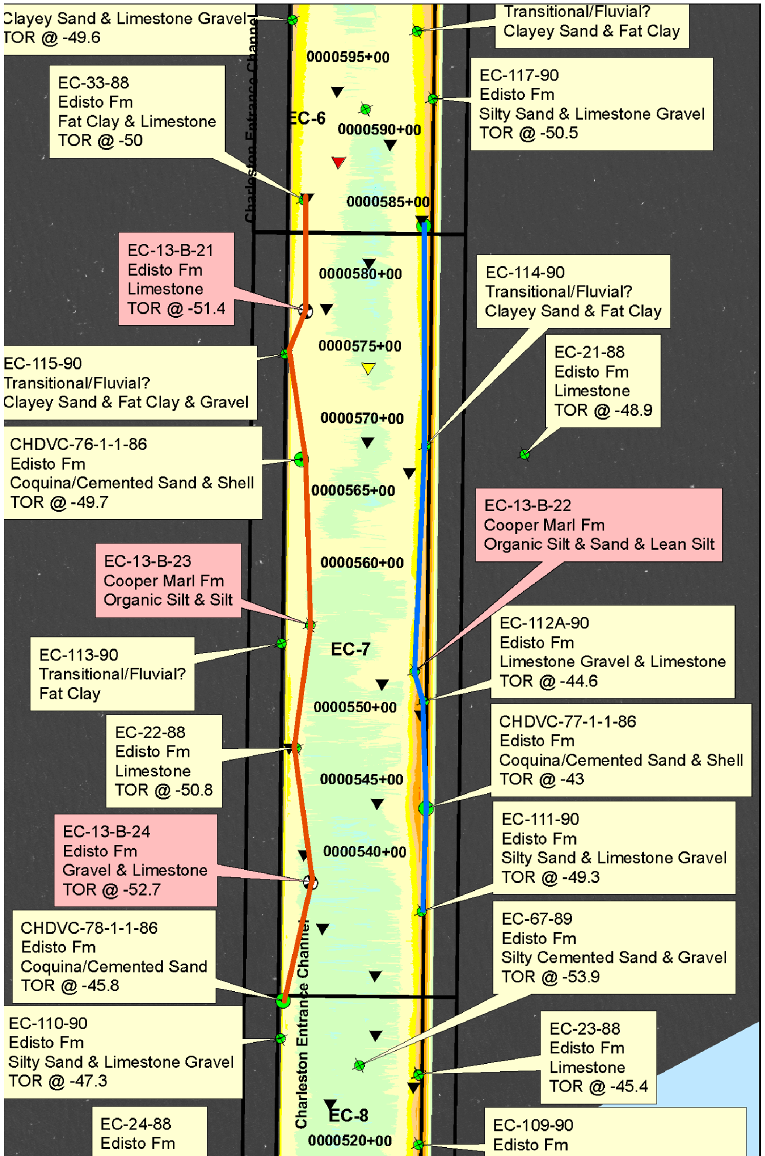


Figure B-56: Harbor Stratigraphy Entrance Channel, Segment EC-7

0 500 1,000 2,000 3,000 Feet

NOTE: Bathymetric color-contoured surface is based upon single-beam sonar condition survey, conducted by CESAC on 25JUN2013.

Legend

Borings
*2013 Boring Info Colored Pink
Rock Core
SPT
Vibracore

Northern Profile

Southern Profile

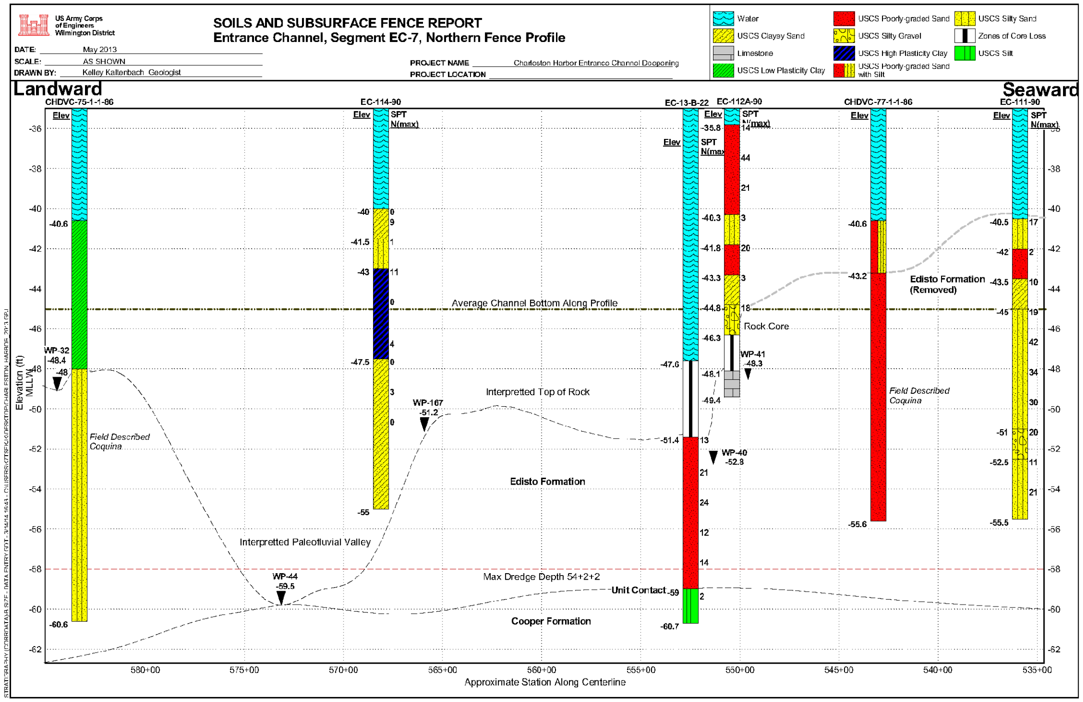
Channel Segments

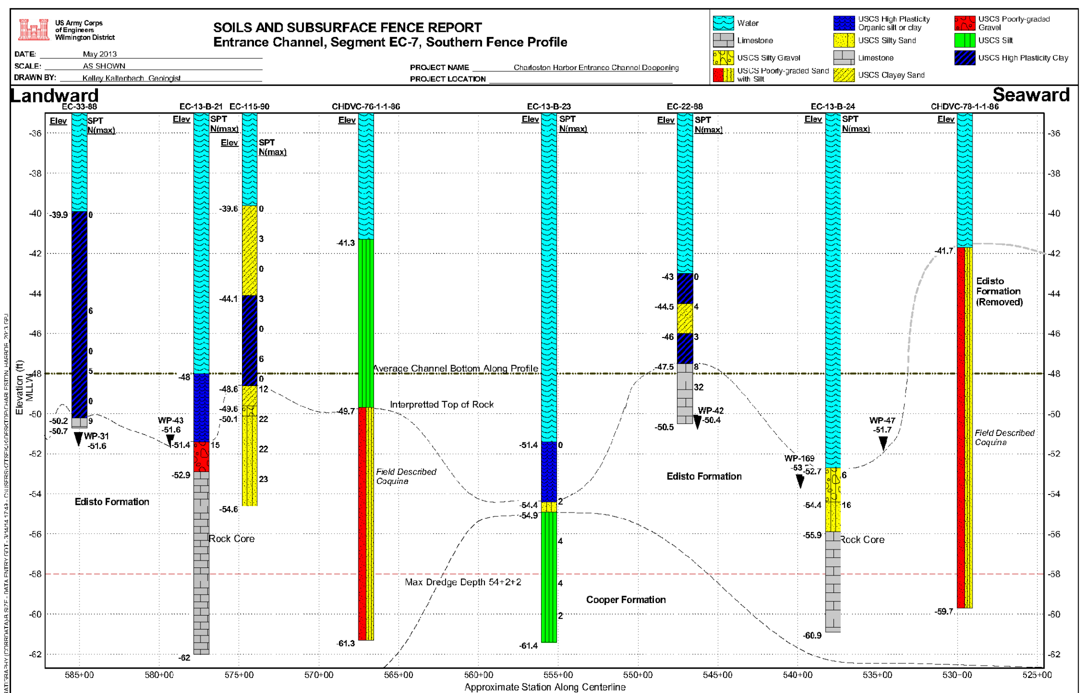
Athena 2013 Washprobes
Refusal within Dredge Prism
Below -56 MLLW
No Refusal Encountered
Poss Rock Encountered

Charleston Harbor Entrance Channel Color-Coded Bathymetry (MLLW)

25.2 - 30	44 - 46	56 - 58
30 - 34	46 - 48	58 - 60
34 - 38	48 - 52	60 - 65
38 - 40	52 - 54	65 - 70
40 - 44	54 - 56	70 - 75

Map Scale: 1:9,000





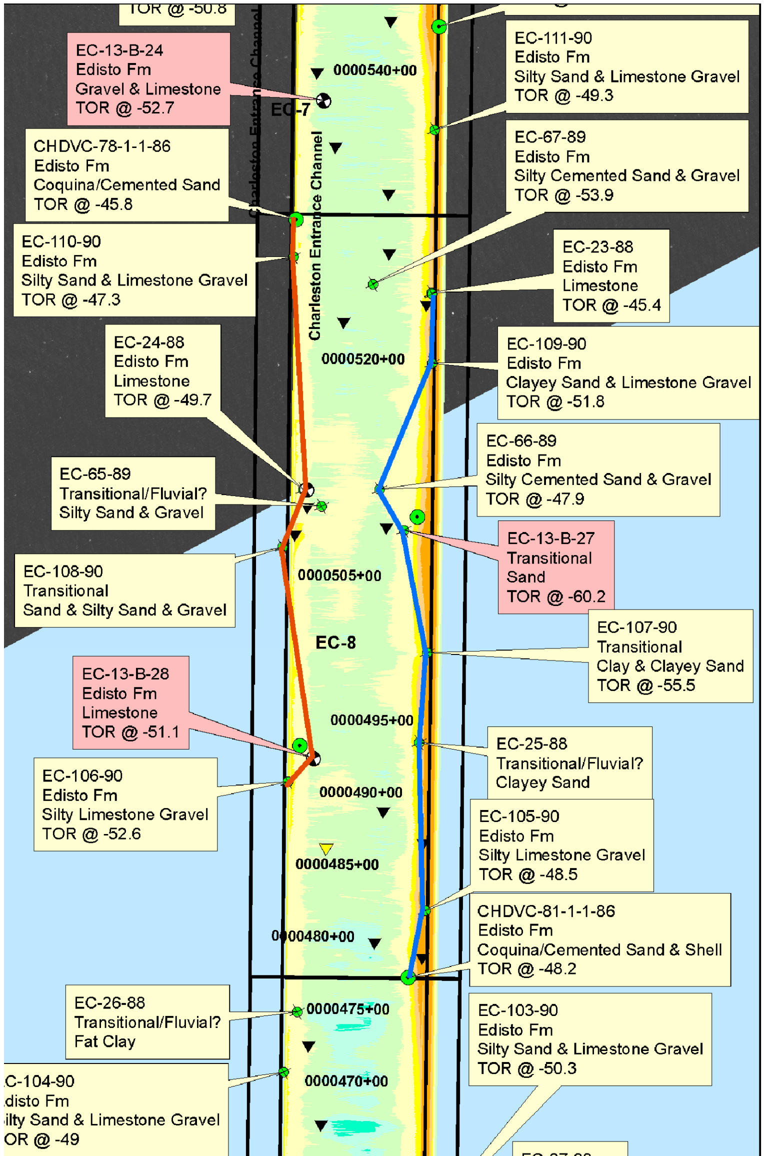
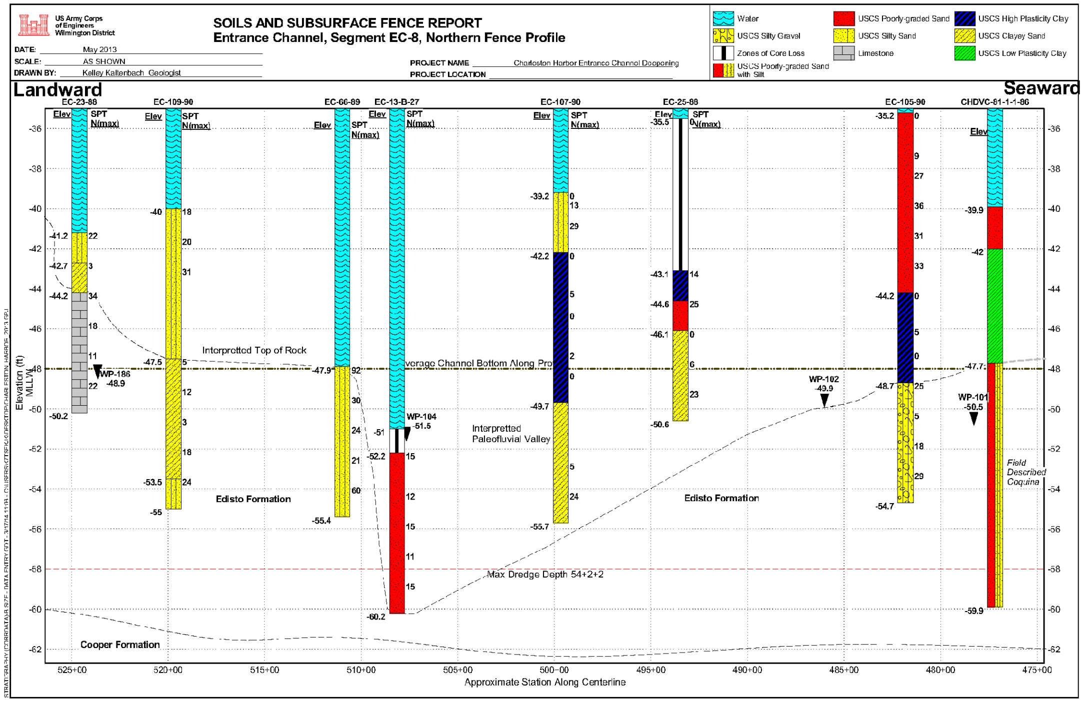


Figure B-57: Harbor Stratigraphy Entrance Channel, Segment EC-8

NOTE: Bathymetric color-contoured surface is based upon single-beam sonar condition survey, conducted by CESAC on 25JUN2013.



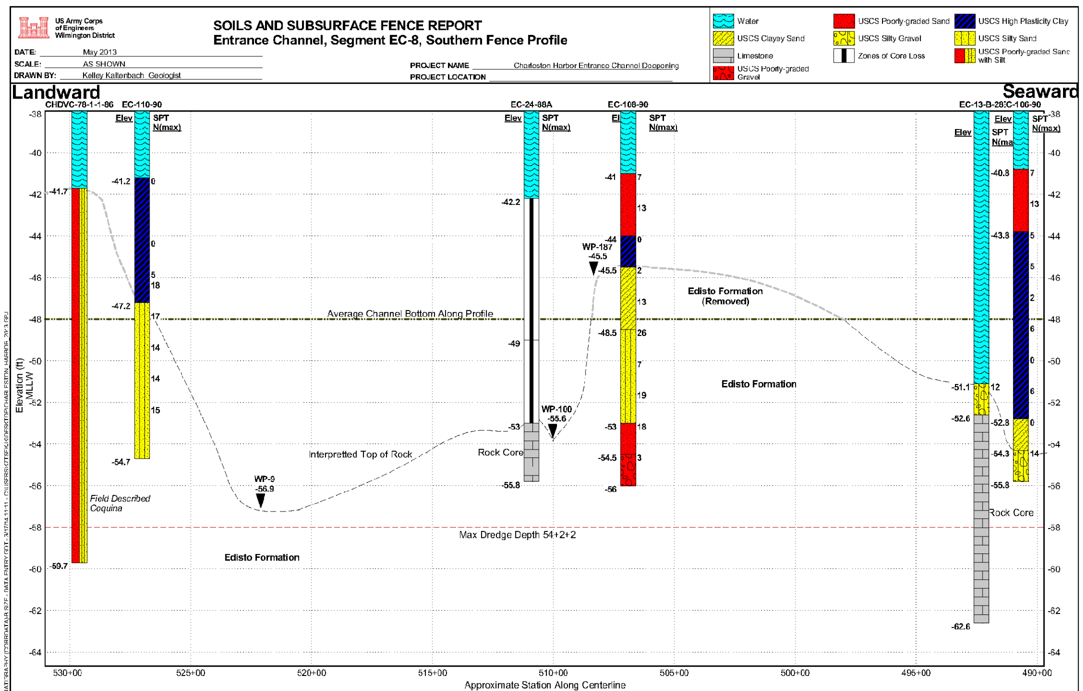
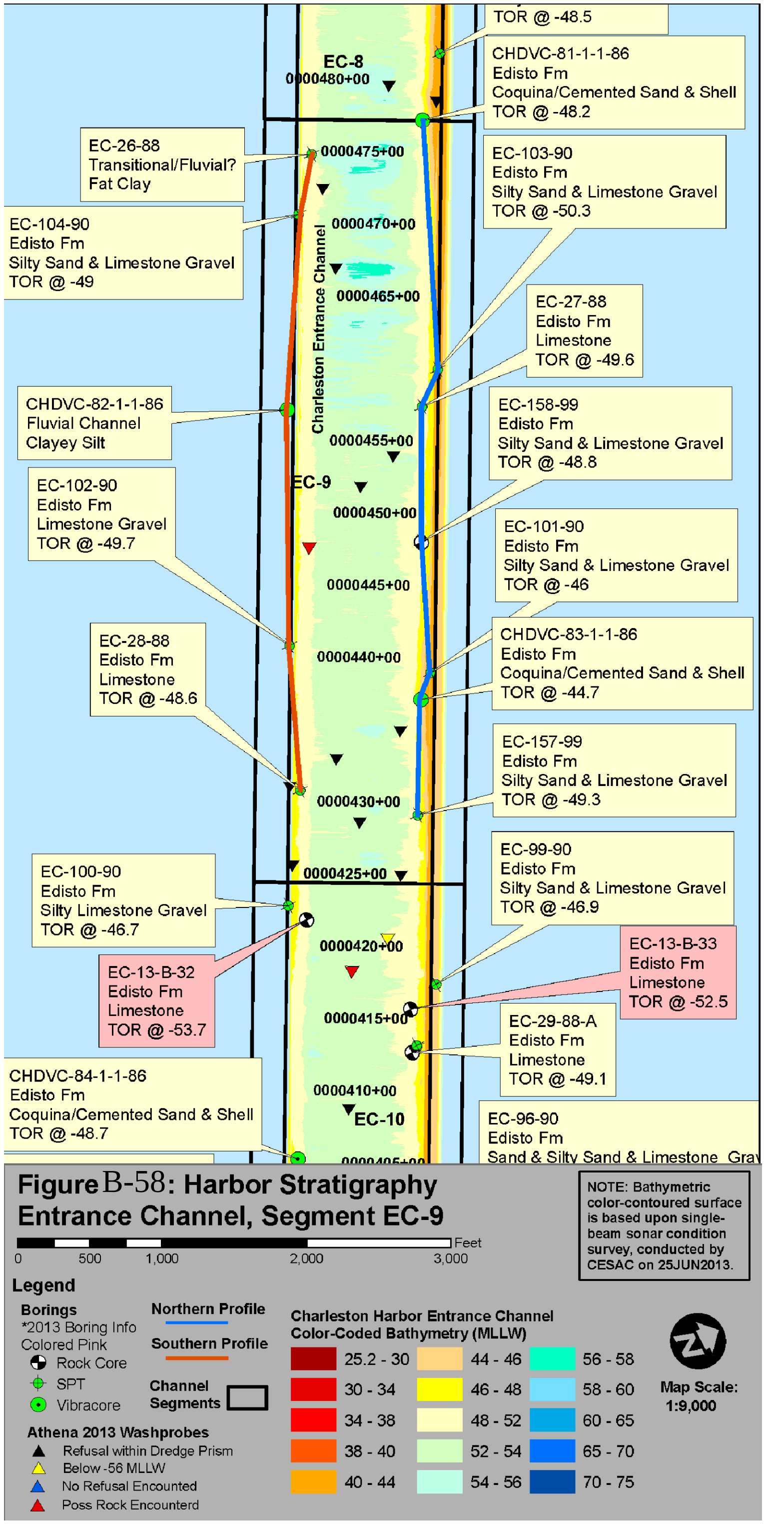
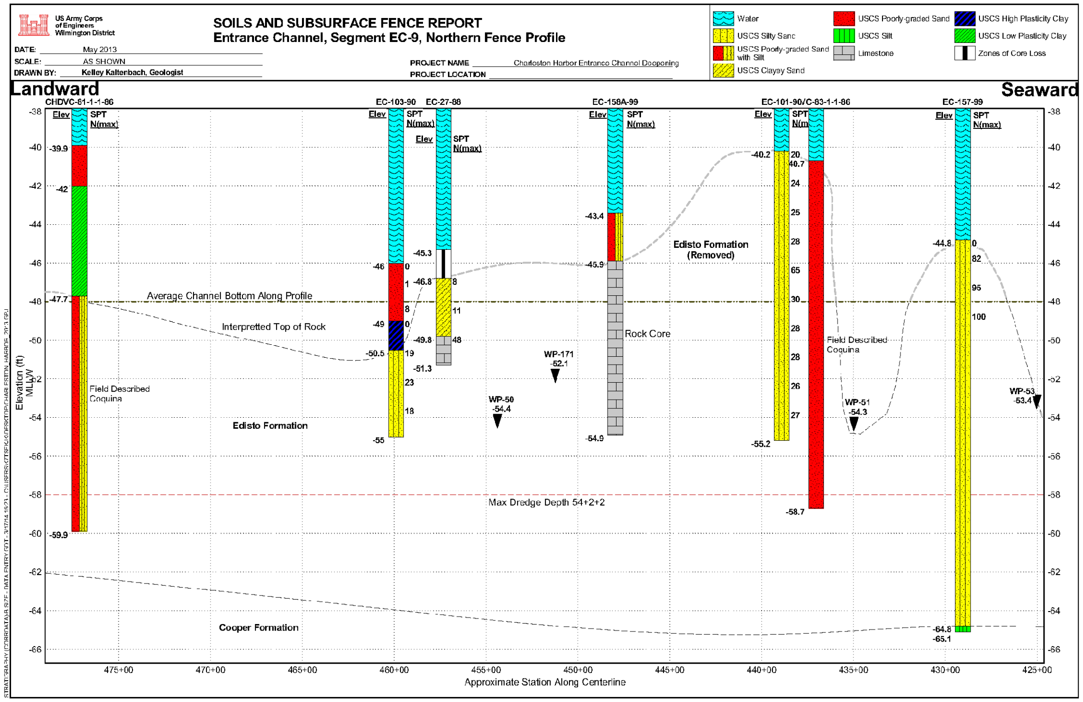
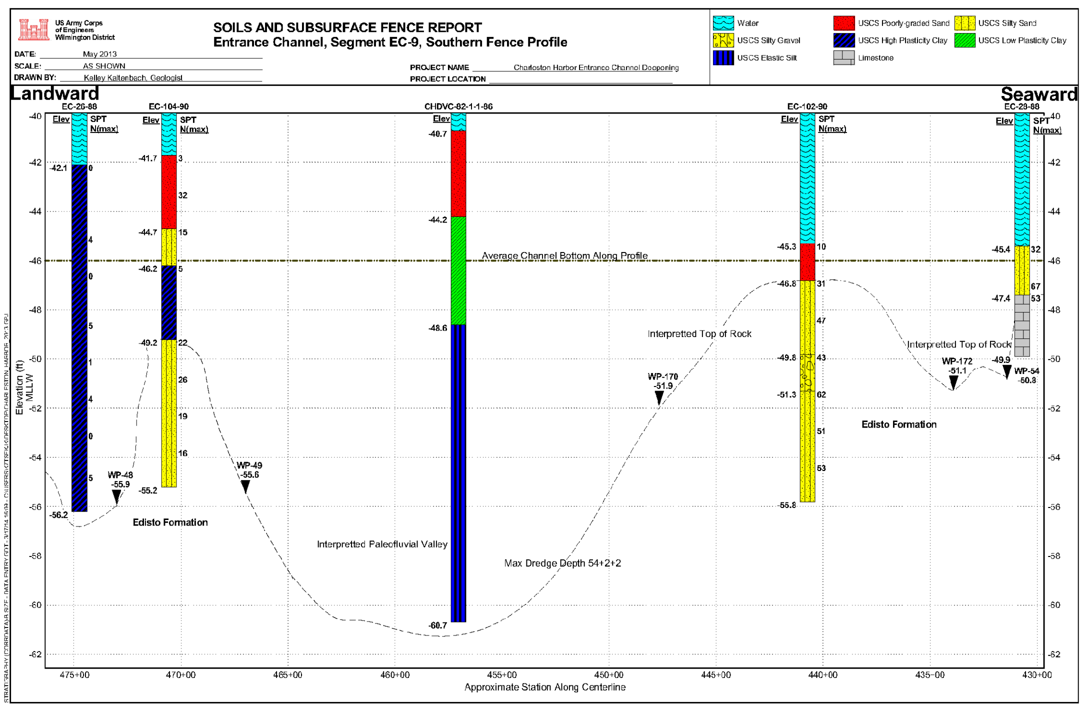


Figure B-58: Harbor Stratigraphy Entrance Channel, Segment EC-9







along the southern profile is -46 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-9 based upon the description of materials in borings CHDVC-81-1-1-86, EC-103-90, EC-27-88, EC-158A-99, EC-101-90, CHDVC-83-1-1-86, EC-104-90, EC-102-90 and EC-28-88 which penetrate the dredge prism to a maximum depth of -64 feet MLLW. Of these borings, only EC-158A-99 is a rock core that sampled intact limestone. The remainder of the borings was advanced by SPT or vibracore. Within the proposed dredging prism, the Edisto Formation has been characterized as coquina, silty calcareous sand, cemented sand with limestone gravel, limestone gravel, or limestone. SPT N-values from borings drilled into this unit indicate that the granular material within the dredging prism is generally medium dense to very dense. Boring data from CHDVC-82-1-1-86 suggests that there may be a buried paleofluvial valley between stations 470+00 and 445+00 on the south side of EC-9. There are no similar features found along the northern profile. The available subsurface data indicates that limestone bedrock will be encountered within the proposed dredging prism for much of the length of segment EC-9. The top of limestone bedrock surface is considered to coincide with the existing bathymetric surface. The exception to this would be the subsurface vicinity of the paleofluvial channel located between stations 470+00 and 445+00, where the top of rock surface is projected below the existing average bathymetric surface.

5.7.10. Entrance Channel, Segment EC-10

Seventeen (17) borings and 1 washprobe were selected from a consolidated gINT dataset of 445 point data to describe the subsurface conditions within segment EC-10 in cross-sectional profile, as shown in Figure B-59. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth ranges from -46 to -54 feet MLLW. The average depth along the northern fence profile is -44.0 feet MLLW, while the average depth along the southern profile is -50 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-10 based upon the description of materials in all of the borings drilled within EC-10. Intact limestone rock cores were recovered from borings EC-13-B-33, EC-29-88A, EC-13-B-36, EC-13-B-37, EC-13-B-32, EC-13-B-34 and EC-13-B-35. The Edisto Formation may extend to depths greater than -64.0 feet based upon existing drilling logs. The remaining borings that were advanced by SPT or vibracore characterize the unit as consisting of coquina, silty calcareous sand, cemented sand with limestone gravel, or as sand with gravel. SPT N-values indicate that the material within the dredging prism are generally medium dense to very dense. The available subsurface data indicates that limestone bedrock will be encountered within much of the proposed dredging prism from station 425+00 to station 370+00. The top of limestone bedrock surface is considered to coincide with the existing bathymetric surface.

5.7.11. Entrance Channel, Segment EC-11

Fourteen (14) borings and 8 washprobes were selected from a consolidated gINT dataset of 445 point data to describe the subsurface conditions within segment EC-11 in cross-sectional profile, as shown in Figure B-60. Single beam sonar condition survey dated 25JUN13 indicates that the channel depth ranges from -46 to -54 feet MLLW. The average depth along both northern and southern fence profiles is -48 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-11 based upon the description of materials in all of the borings drilled within the channel segment. Intact limestone rock cores were recovered from borings EC-13-B-39, EC-13-B-41, EC-13-B-43, EC-13-B-38, EC-13-B-40, EC-87-89 and EC-13-B-42. The Edisto Formation may extend to depths greater than -69.0 feet based upon the existing drilling logs. The remaining borings that were advanced by SPT or vibracore characterize the unit as consisting of coquina, silty calcareous sand, and cemented sand with limestone gravel. SPT N-values indicate that the limestone is generally soft and weakly cemented, and that the material within the dredging prism are generally medium dense to very dense. The available subsurface data indicates that limestone bedrock will be encountered throughout much of the proposed dredging prism from station 370+00 to station 320+00. The top of limestone bedrock surface is considered to coincide with the existing bathymetric surface. Potential exception to this is the presence of two small valley or trough features that are located between stations 330+00 and 325+00 along the northern side of the channel, and between stations 355+00 and 345+00 on the southern side. The degree to which these features are in-filled with unconsolidated sediment (if at all) is unknown.

5.7.12. Entrance Channel, Segment EC-12

Eleven (11) borings and 1 washprobe were selected from a consolidated gINT dataset of 445 point data to describe the subsurface conditions within segment EC-12 in cross-sectional profile, as shown in Figure B-61. Single beam sonar condition survey dated 25JUN13 indicates that much of the channel depth ranges from -48 to -54 feet MLLW. The average depth along the northern fence profile is -48 feet MLLW, while the southern fence profile is deeper at -53 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-12 based upon the description of materials in all of the borings drilled within the channel segment. Intact limestone rock cores were recovered from borings EC-13-B-45, EC-13-B-47, EC-13-B-49, EC-13-B-44, EC-13-B-46 and EC-13-B-48. The Edisto Formation extends to depths greater than -62.0 feet MLLW based upon the existing drilling logs. Borings that were advanced by SPT or vibracore characterize the unit as consisting of coquina, silty calcareous sand, and cemented sand with some limestone gravel. These materials are directly correlated to the limestone recovered in the adjacent rock cores. SPT N-values indicate that the limestone is generally soft and weakly cemented, and that the material within the dredging prism are generally medium dense to very dense. The available subsurface data indicates that limestone bedrock will be encountered throughout much of the proposed dredging prism from station 311+00 to station 280+00. The top of limestone bedrock surface is considered to coincide with the existing bathymetric surface.

5.7.13. Entrance Channel, Segment EC-13

Seven (7) borings and 6 washprobes were selected from a consolidated gINT dataset of 445 point data to describe the subsurface conditions within segment EC-13 in cross-sectional profile, as shown in Figure B-62. An underwater photograph taken from a washprobe shows the general nature of the ocean bottom in this segment. Boring EC-13-B-54 was used for each profile in order to extend the length of the fence diagrams within EC-13. Single beam sonar condition survey dated 25JUN13 indicates that much of the channel depth ranges from -48 to -52 feet MLLW, with occasional troughs that have depths up to -54 feet MLLW. The average depth along the northern fence profile is -48 feet MLLW, while the southern fence profile is deeper at -

50 feet MLLW. The maximum proposed dredge depth is -58 feet MLLW. Variations in the bathymetric depth along profile are not shown. The Edisto Formation is the predominant lithologic unit within EC-13, which is covered by a relatively thin veneer of sandy to gravelly material based upon the 2013 borings and underwater photograph (Figure B-62). Intact limestone rock cores with high amounts of recovery were recovered from borings EC-13-B-50, EC-13-B-52, and EC-13-B-51. Borings EC-13-B-53 and EC-13-B-54 encountered quartz sands that appeared to overlie sand mixed with weakly cemented limestone gravel. The lack of cementation in the quartz sand may indicate either a facies change within the Edisto Formation, or a poorly defined lithologic boundary between the limestone of the Edisto Formation, and the sands of the Marks Head Formation. Washprobe refusal depths seems to indicate that there is a distinctly denser surface at -52.7 to -52.8 feet MLLW, which corresponds with depth to which the limestone gravel occurs in borings EC-13-B-53 and EC-13-B-54. Therefore, the top of rock surface for the Edisto Formation is considered to lie at -52.7 feet MLLW, which is stratigraphically overlain by the medium dense sands of the Marks Head Formation. This stratigraphic positioning of units is consistent with the work of Weems and Lemon (1993), and projects the top of the Edisto Formation to gently plunge into the subsurface with increasing distance seaward. SPT N-values taken within the Edisto Formation indicate that the limestone is weakly cemented and has medium density against penetration. The sands of the Marks Head Formation, present from station 225+00 seaward are also medium dense. The available subsurface data indicates that limestone bedrock will be encountered within the proposed dredging prism from station 260+00 to at least station 210+00; however, the top of limestone bedrock surface will likely plunge from the existing bathymetric surface to -54.5 feet MLLW, and continue into the subsurface further offshore.

5.7.14. Entrance Channel, Segment EC-14

Two (2) borings and 7 washprobes were selected from a consolidated gINT dataset of 445 point data to describe the subsurface conditions within segment EC-14 in cross-sectional profile, as shown in Figure B-64. The lack of borings within EC-14 limits the length and control by which fence diagrams can be drafted. Washprobes between the two borings were used to provide vertical control on the interpreted top of rock surface. Single beam sonar condition survey dated 25JUN13 indicates that much of the channel depth ranges from -48 to -54 feet MLLW. The average depth along both northern and southern fence profiles is -51.5 feet MLLW. Variations in the bathymetric depth along profile are not shown. Borings EC-13-B-54 and EC-13-B-55 encountered weakly cemented sand and limestone gravel at -54.9 and -55.6 respectively. Nearby washprobes WP-129, WP-202, WP-131, WP-203 and WP-127 have similar refusal depths that range between -54 to -56 MLLW. This suggests there is a dense cemented horizon that corresponds to the gravelly strata in borings EC-13-B-54 and EC-13-B-55. Therefore, the top of rock surface for the Edisto Formation is considered to lie between -54 and -56 feet MLLW within EC-14. Overlying the Edisto Formation is a medium dense, poorly graded quartz sand that grades seaward into an interbedded sequence of sand and silt, as shown in the borings. This material is tentatively considered part of the Marks Head Formation, based largely on the work of Weems and Lemon (1993). Little is known of this material between the two available borings EC-13-B-54 and EC-13-B-55. SPT N-values indicate that material within the dredging prism is weakly cemented and medium dense to dense. The horizontal extent of the strata is not well constrained because there are only two borings available for interpolation.

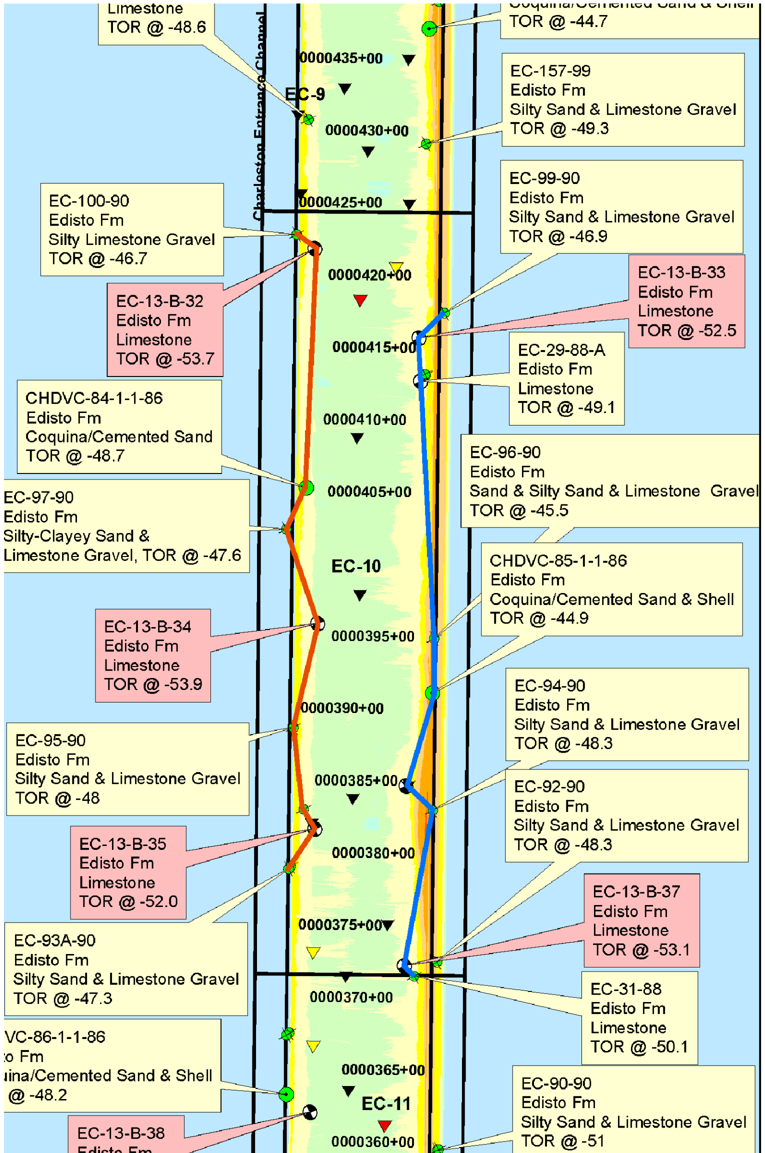
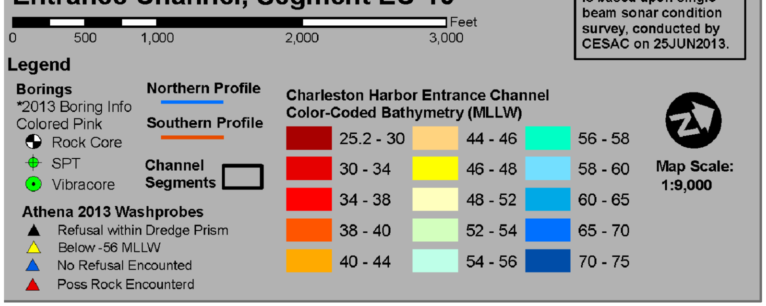
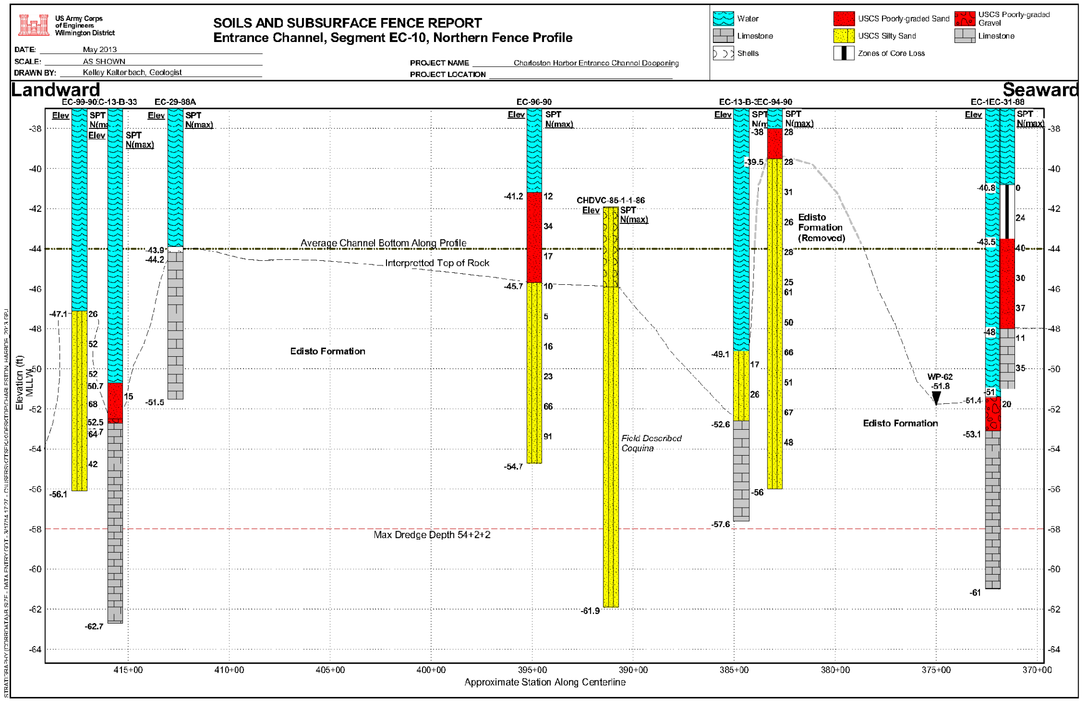
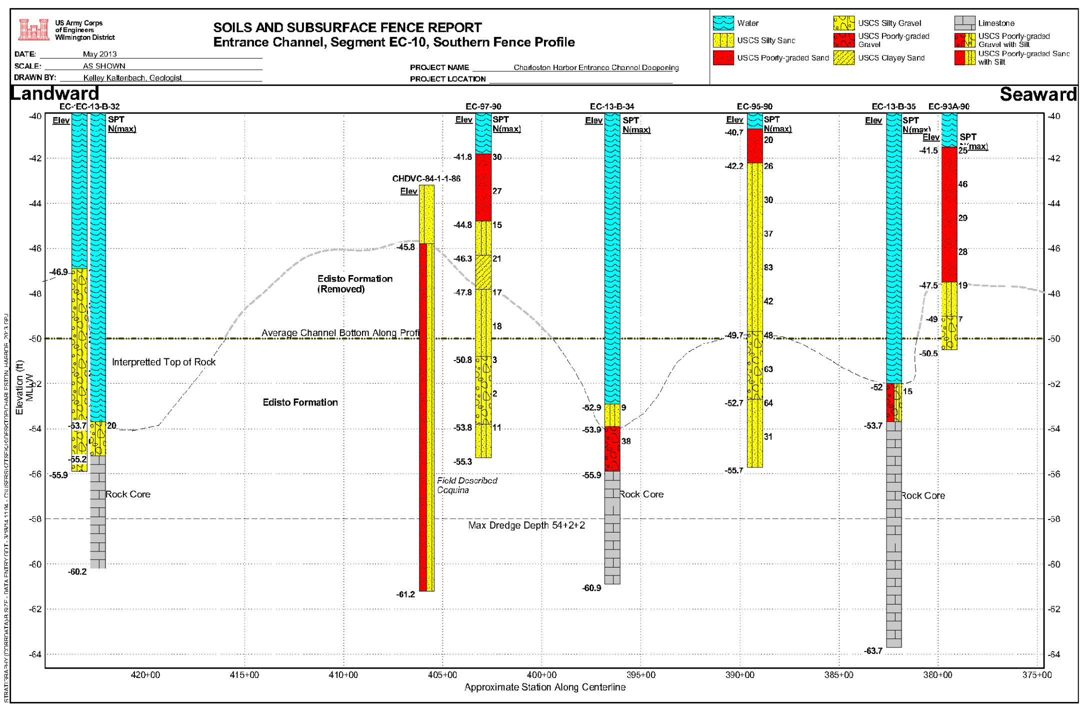


Figure B-59: Harbor Stratigraphy Entrance Channel, Segment EC-10

NOTE: Bathymetric color-contoured surface is based upon single-beam sonar condition survey, conducted by CESAC on 25JUN2013.







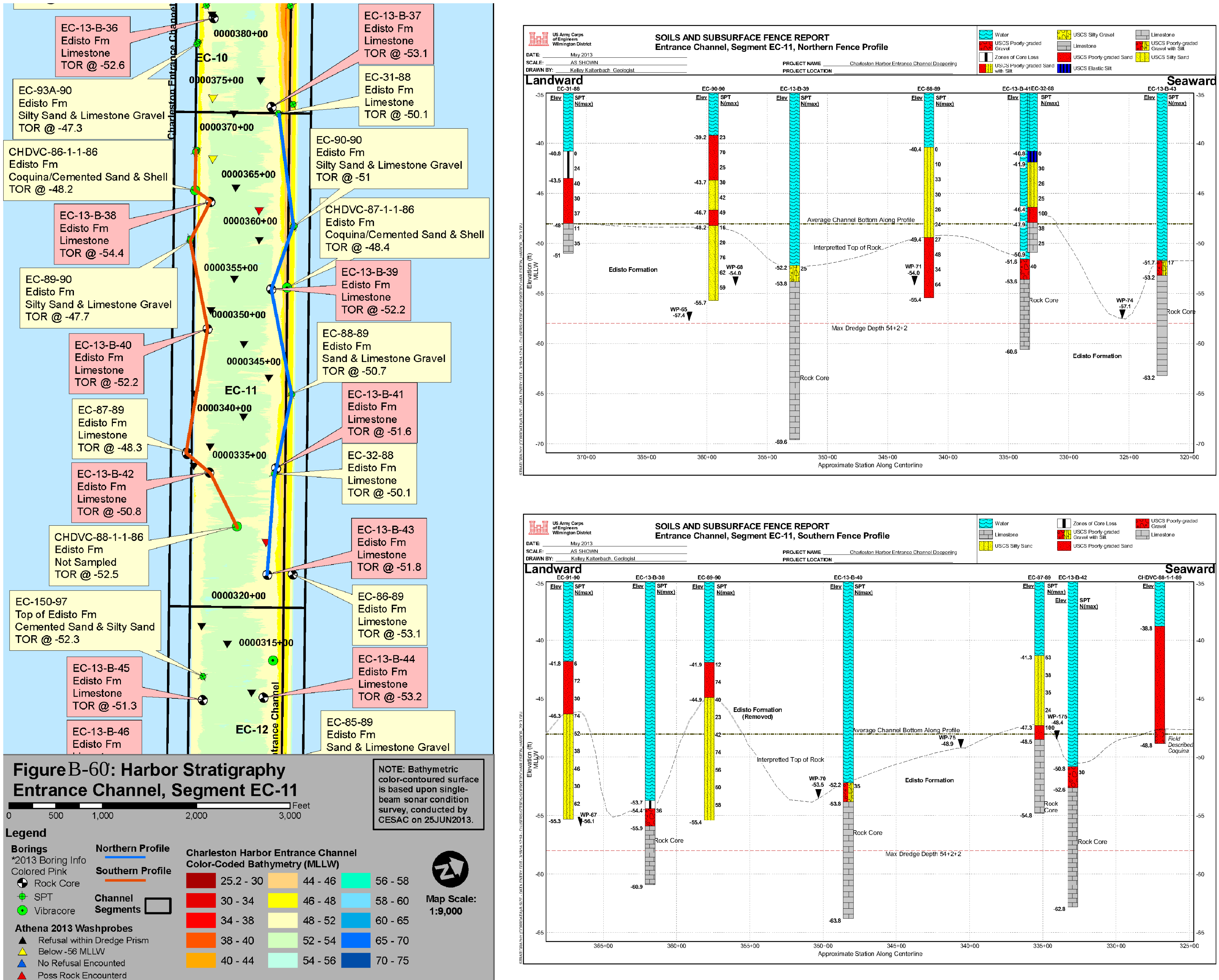


Figure B-60: Harbor Stratigraphy Entrance Channel, Segment EC-11

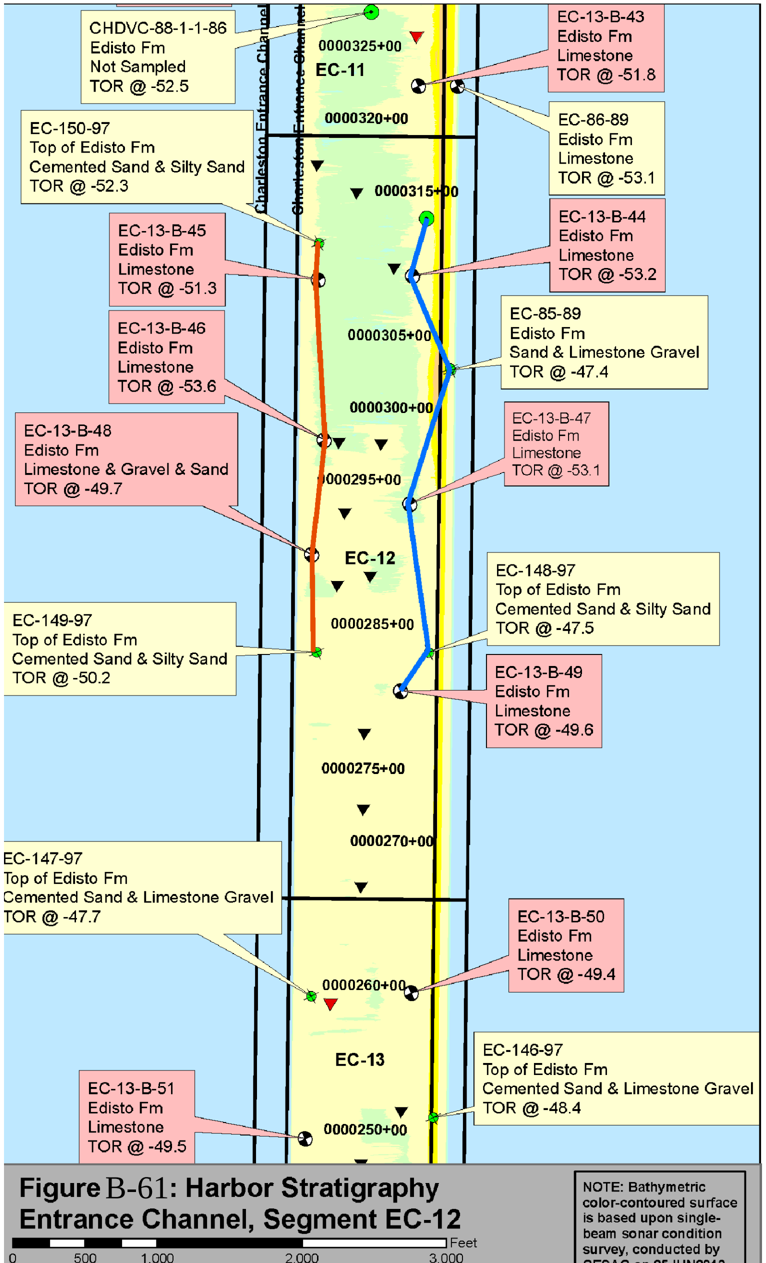
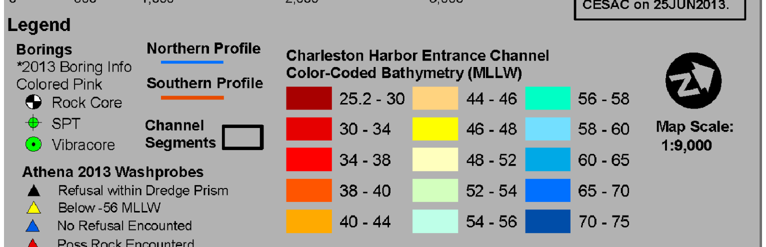
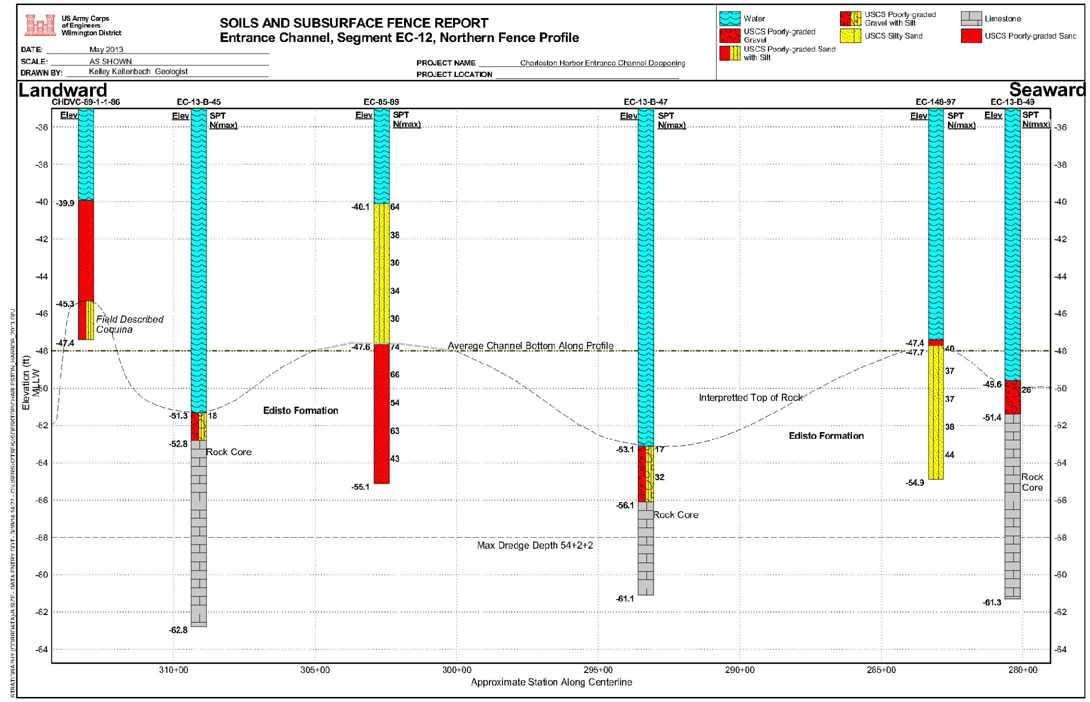
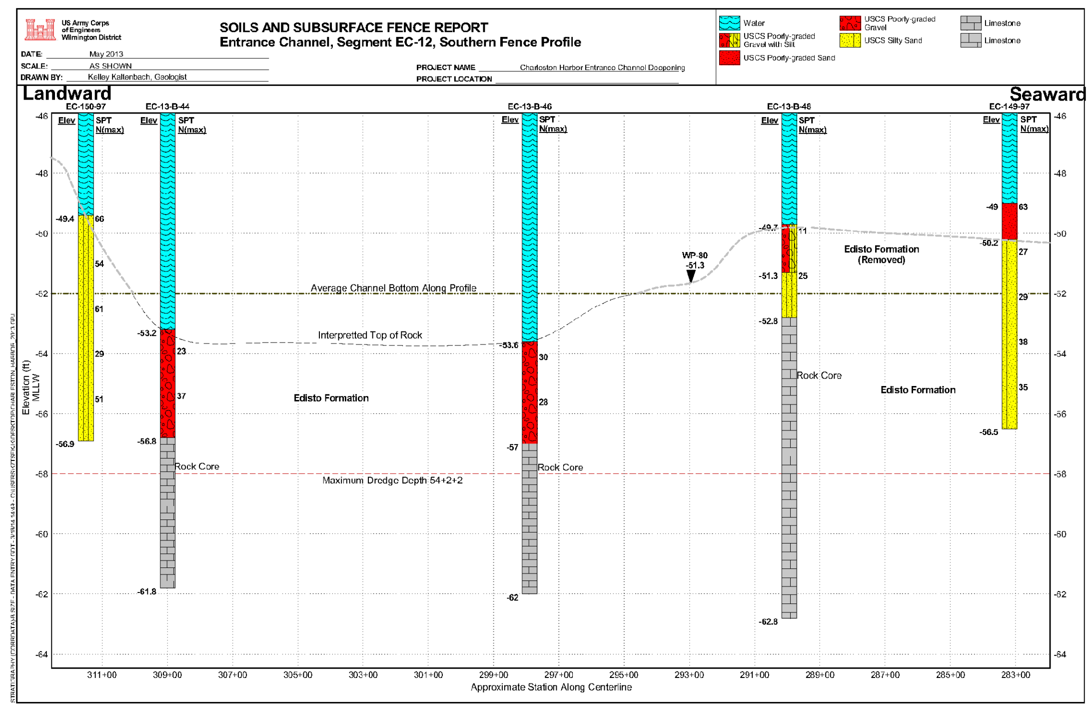


Figure B-61: Harbor Stratigraphy Entrance Channel, Segment EC-12

NOTE: Bathymetric color-contoured surface is based upon single-beam sonar condition survey, conducted by CESAC on 25JUN2013.







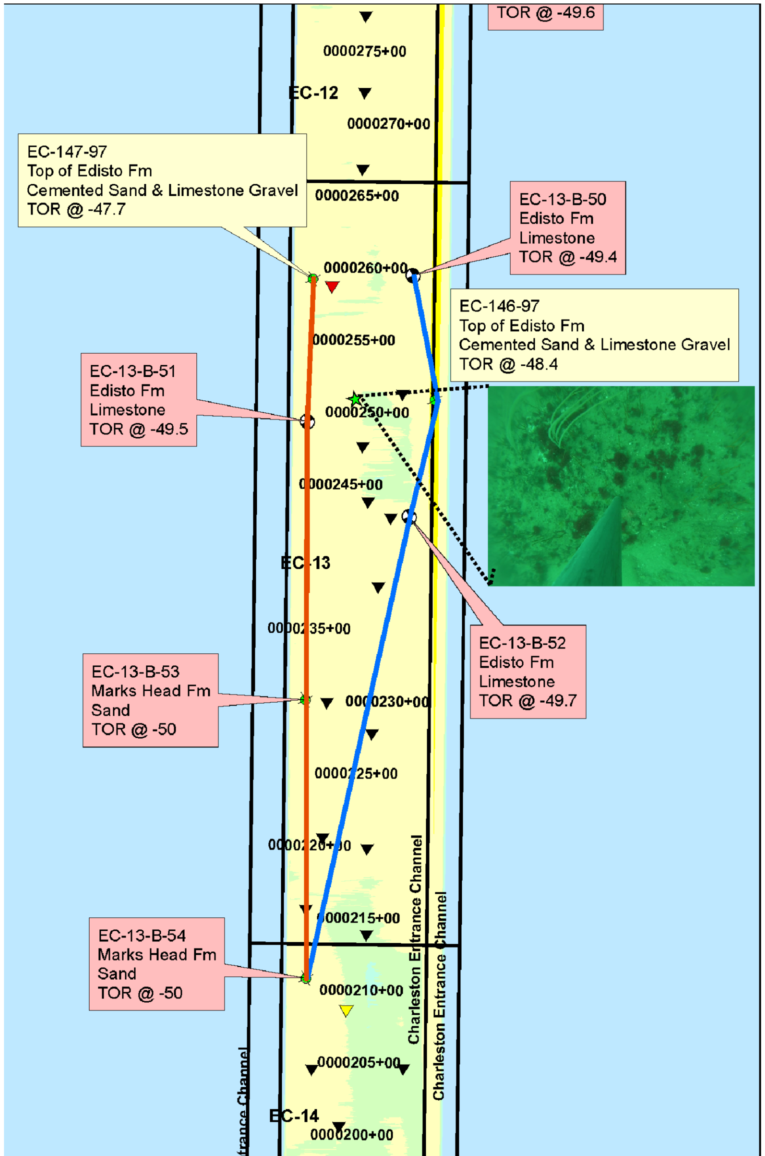


Figure B-62: Harbor Stratigraphy Entrance Channel, Segment EC-13

0 500 1,000 2,000 3,000 Feet

NOTE: Bathymetric color-contoured surface is based upon single-beam sonar condition survey, conducted by CESAC on 25JUN2013.

Legend

Borings
*2013 Boring Info Colored Pink
- Rock Core
- SPT
- Vibracore

Northern Profile

Southern Profile

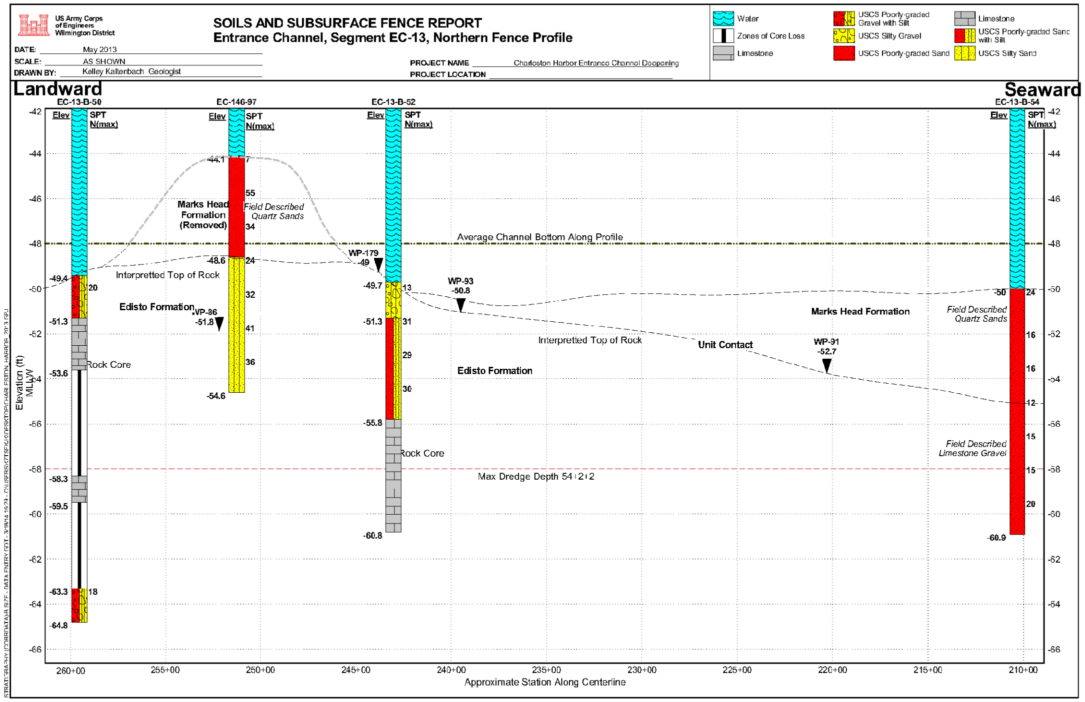
Channel Segments

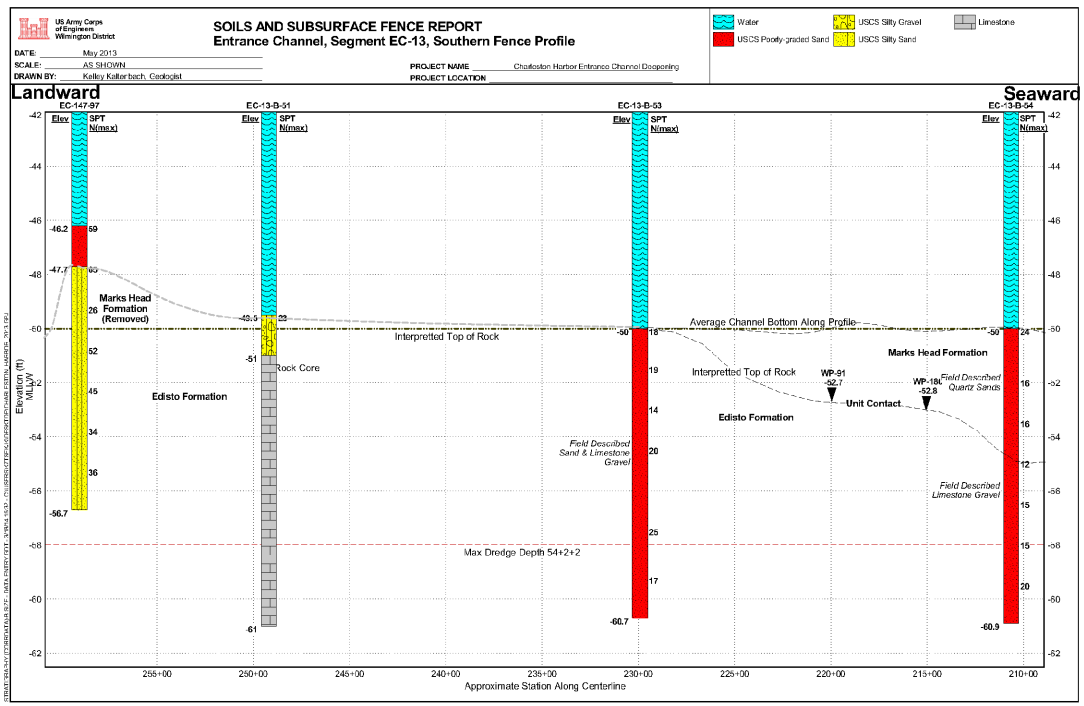
Athena 2013 Washprobes
- Refusal within Dredge Prism
- Below -56 MLLW
- No Refusal Encounted
- Poss Rock Encountered

Charleston Harbor Entrance Channel Color-Coded Bathymetry (MLLW)

25.2 - 30	44 - 46	56 - 58
30 - 34	46 - 48	58 - 60
34 - 38	48 - 52	60 - 65
38 - 40	52 - 54	65 - 70
40 - 44	54 - 56	70 - 75

Map Scale: 1:9,000





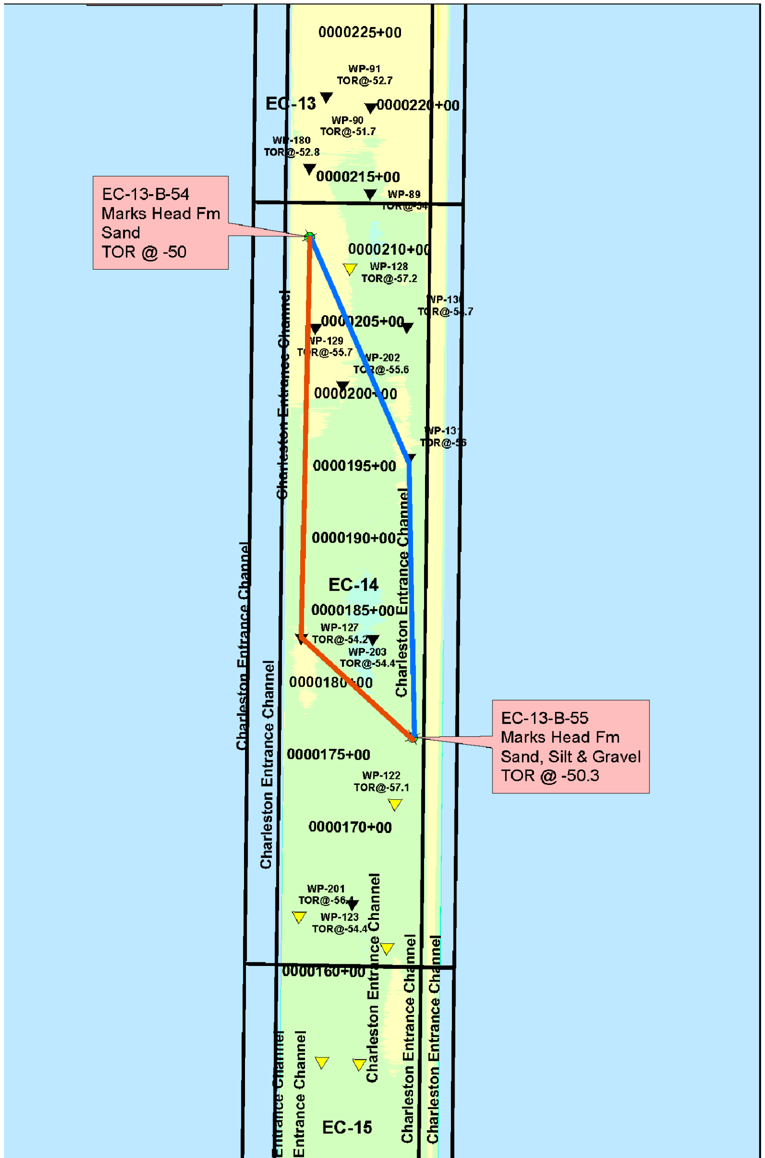


Figure B-63: Harbor Stratigraphy Entrance Channel, Segment EC-14

Feet
0 500 1,000 2,000 3,000

NOTE: Bathymetric color-contoured surface is based upon single-beam sonar condition survey, conducted by CESAC on 25JUN2013.

Legend

Borings
*2013 Boring Info Colored Pink
- Rock Core
- SPT
- Vibracore

Northern Profile

Southern Profile

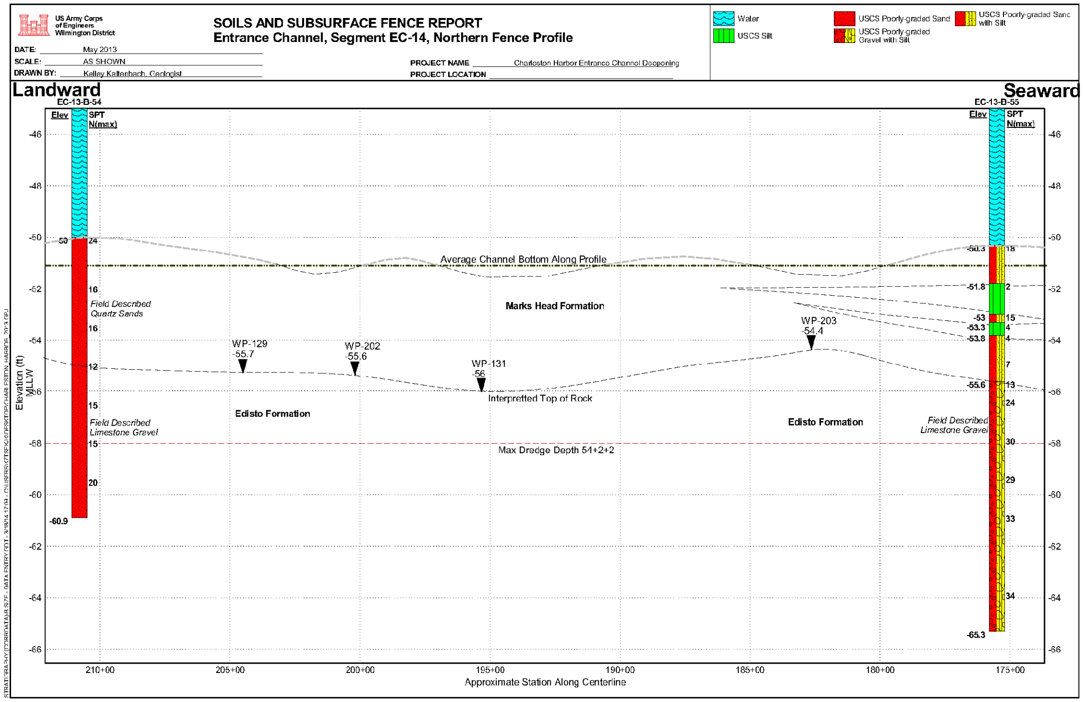
Channel Segments

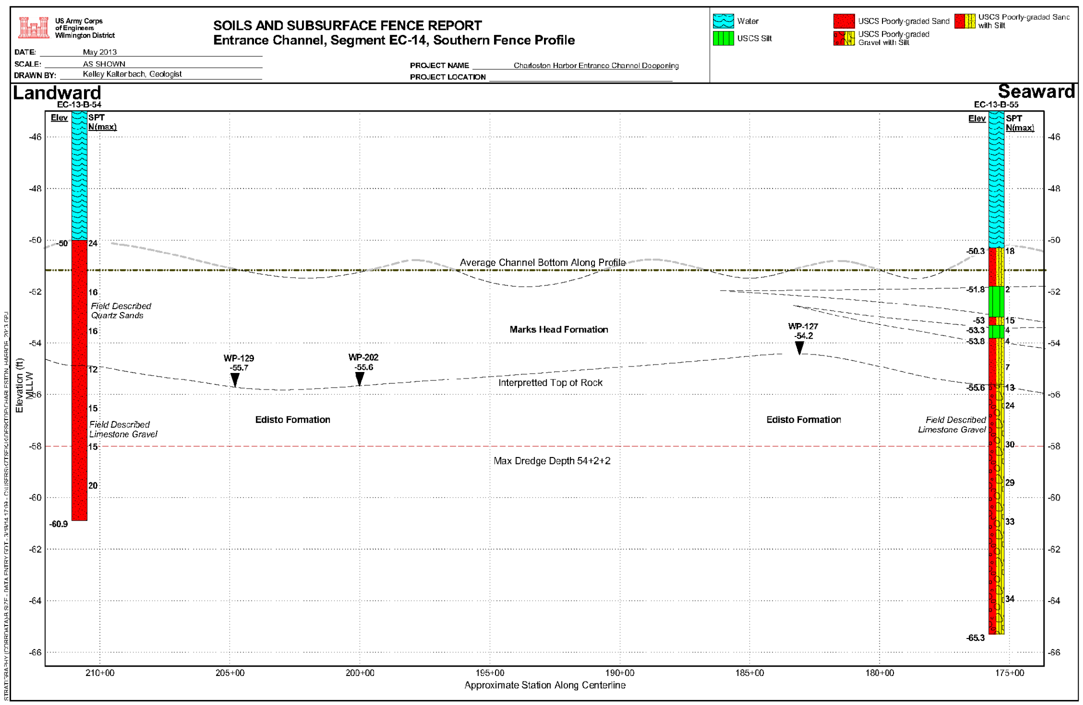
Athena 2013 Washprobes
- Refusal within Dredge Prism
- Below -56 MLLW
- No Refusal Encountered
- Poss Rock Encountered

Charleston Harbor Entrance Channel Color-Coded Bathymetry (MLLW)

25.2 - 30	44 - 46	56 - 58
30 - 34	46 - 48	58 - 60
34 - 38	48 - 52	60 - 65
38 - 40	52 - 54	65 - 70
40 - 44	54 - 56	70 - 75

Map Scale: 1:9,000





5.7.15. Entrance Channel, Segment EC-15

Two (2) borings and 13 washprobes were selected from a consolidated gINT dataset of 445 point data to describe the subsurface conditions within segment EC-15 in cross-sectional profile, as shown in Figure B-59. The lack of borings within EC-15 required the use of borings EC-13-B-55 and EC-145-97, which are located within adjacent channel segments, in order to effectively draft the fence diagrams for Figure B-65. Vertical control on the interpreted top of rock surface was augmented by the relatively abundant number of washprobes in EC-15. Single beam sonar condition survey dated 25JUN13 indicates that much of the channel depth ranges from -52 to -54 feet MLLW. The average depth along both northern and southern fence profiles is -52.0 feet MLLW. Variations in the bathymetric depth along profile are not shown. Boring and washprobe data suggests that the top of the Edisto Formation dips below the proposed dredging prism near station 160+00 and plunges deeper into the subsurface with increasing distance seaward. The overlying interbedded sequence of silt and sand strata, presumably part of the Marks Head Formation, appears to grade laterally into a thick bed of fat clay, bases upon material sampled in boring EC-145-97. It is not known if this material represents a facies change within the Marks Head Formation or an in-filled paleo-fluvial channel. There are no SPT N-values between the two borings in Figure B-59, however washprobe refusal is well below the maximum proposed dredge depth seaward of station 160+00, which indicates that the in-situ material is weak and can be easily removed.

5.7.16. Entrance Channel, Segment EC-16

Five (5) borings and 9 washprobes were selected from a consolidated gINT dataset of 445 point data to describe the subsurface conditions within segment EC-16 in cross-sectional profile, as shown in Figure B-65. Boring EC-145-97 was used as a common starting point for drafting the two fence diagrams. Vertical control on the interpreted top of rock surface was augmented by the adjacent washprobes. Single beam sonar condition survey dated 25JUN13 indicates that much of the channel depth ranges from -48 to -58 feet MLLW. The average depth along both northern and southern fence profiles is -51.0 feet MLLW. Variations in the bathymetric depth along profile are not shown. Boring and washprobe data suggests that the top of the Edisto Formation is irregular and hummocky, but is well below the maximum proposed dredge depth of -58 feet MLLW. The overlying stratum consists of soft fat clay overlain by dense to very dense quartz sand, based upon the SPT borings. The dense to very dense sand occurs near station 85+00 and extends to station 60+00 on the north side of the channel. On the south side of the channel, the sand occurs near station 92+00 and extends to station 64+00. Much of the very dense sand appears to have been removed through previous harbor deepening, however the depth and lateral extent of the material is not well constrained due to the relatively few borings present in the outer channel segments. It is assumed, based upon washprobe refusal data and existing bathymetry that the dense cemented sands are limited in extent and locally comprise the banks on either side of the channel, which lie between the -48 to -52 contours. This material is not as expansive as the limestone of the Edisto Formation, but may require some limited removal by rock cutter head.

5.7.17. Entrance Channel, Segment EC-17

Seven (7) washprobes were selected from a consolidated gINT dataset of 445 point data to illustrate the interpreted top of rock surface within segment EC-17 in cross-sectional profile, as shown in Figure B-66. Single beam sonar condition survey dated 25JUN13 indicates that the channel bottom is extremely varied, having a bathymetric range between -48 to -70 feet MLLW. The average depth along both northern and southern fence profiles is -51.0 feet MLLW. Variations in the bathymetric depth along profile are not shown. Washprobe refusal data indicates that the interpreted top of rock surface lies near -65 feet MLLW, which is well below the maximum proposed dredge depth of -58 feet MLLW. The overlying stratum was penetrated by washprobes, therefore it is assumed that this material is very soft/loose and may be easily removed.

5.7.18. Entrance Channel, Segment EC-18

Six (6) washprobes were selected from a consolidated gINT dataset of 445 point data to illustrate the interpreted top of rock surface within segment EC-18 in cross-sectional profile, as shown in Figure B-67. Single beam sonar condition survey dated 25JUN13 indicates that the channel bottom is extremely varied, having a bathymetric range between -48 to -65 feet MLLW. The average depth along both northern and southern fence profiles is -53.0 feet MLLW. Variations in the bathymetric depth along profile are not shown. Washprobe refusal data indicates that the interpreted top of rock surface lies between -65 and -61 feet MLLW, which is well below the maximum proposed dredge depth of -58 feet MLLW. The overlying stratum was penetrated by washprobes, therefore it is assumed that this material is very soft/loose and may be easily removed.

5.7.19. Entrance Channel, Segment EC-19

Eight (8) washprobes were selected from a consolidated gINT dataset of 445 point data to illustrate the interpreted top of rock surface within segment EC-19 in cross-sectional profile, as shown in Figure B-68. Single beam sonar condition survey dated 25JUN13 indicates that the channel bottom is extremely varied, having a bathymetric range between -48 to -65 feet MLLW. The average depth along both northern and southern fence profiles is -53.0 feet MLLW. Variations in the bathymetric depth along profile are not shown. Washprobe refusal data indicates that the interpreted top of rock surface lies between -64 and -61 feet MLLW, which is well below the maximum proposed dredge depth of -58 feet MLLW. The overlying stratum was penetrated by the washprobes shown in Figure B-63, therefore it is assumed that this material is very soft/loose and may be easily removed.

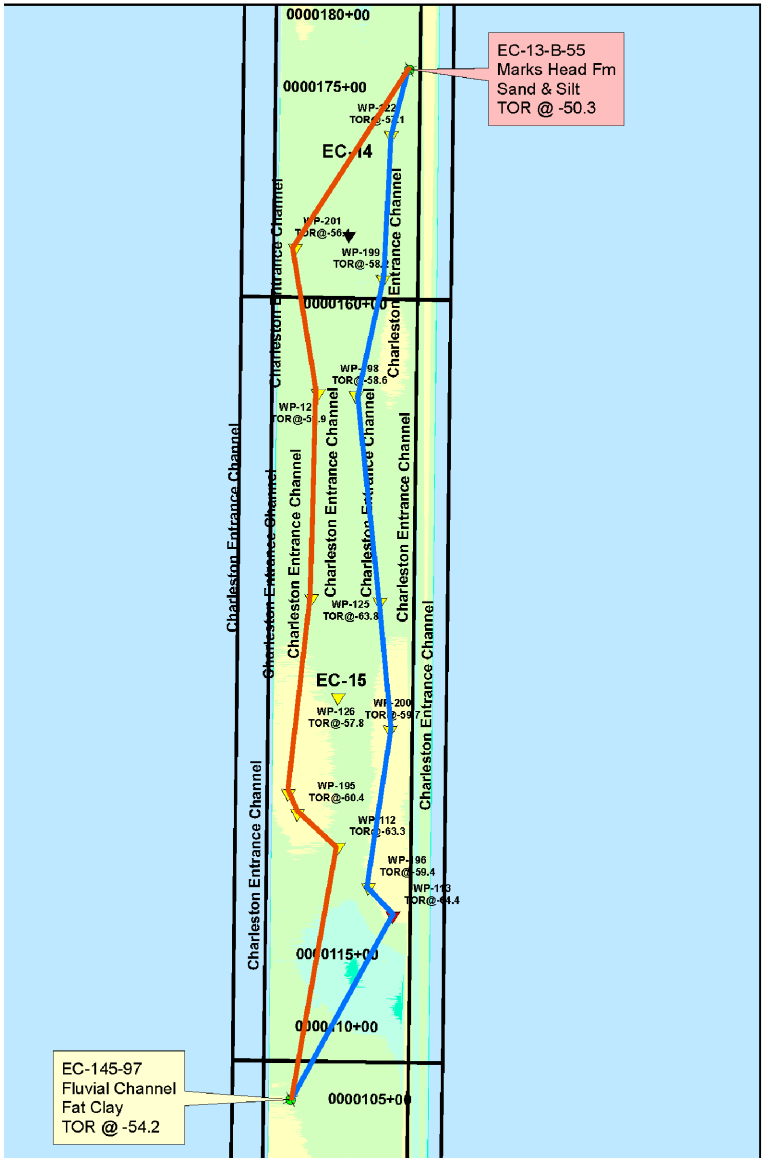
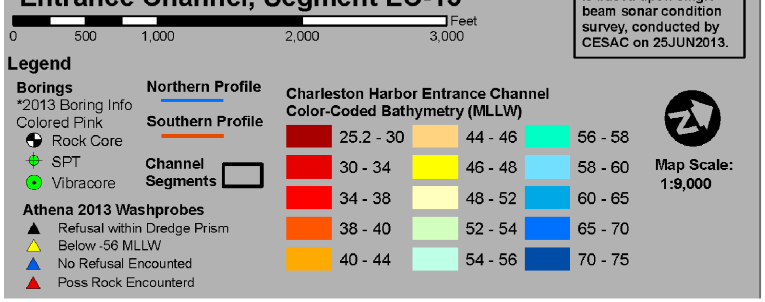
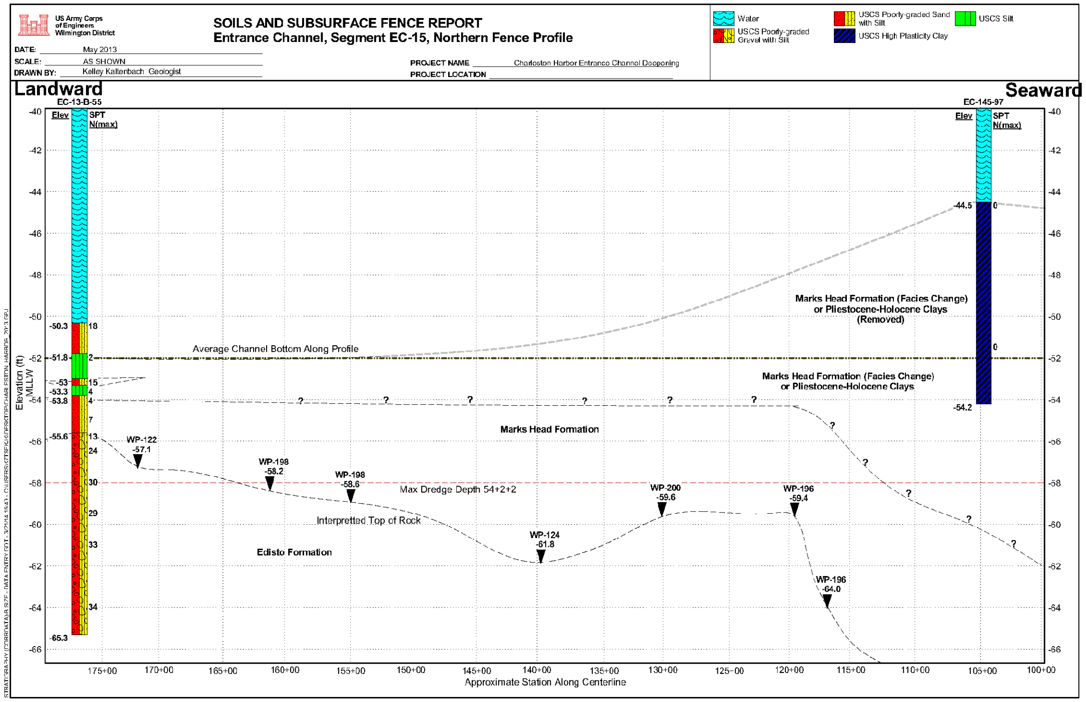
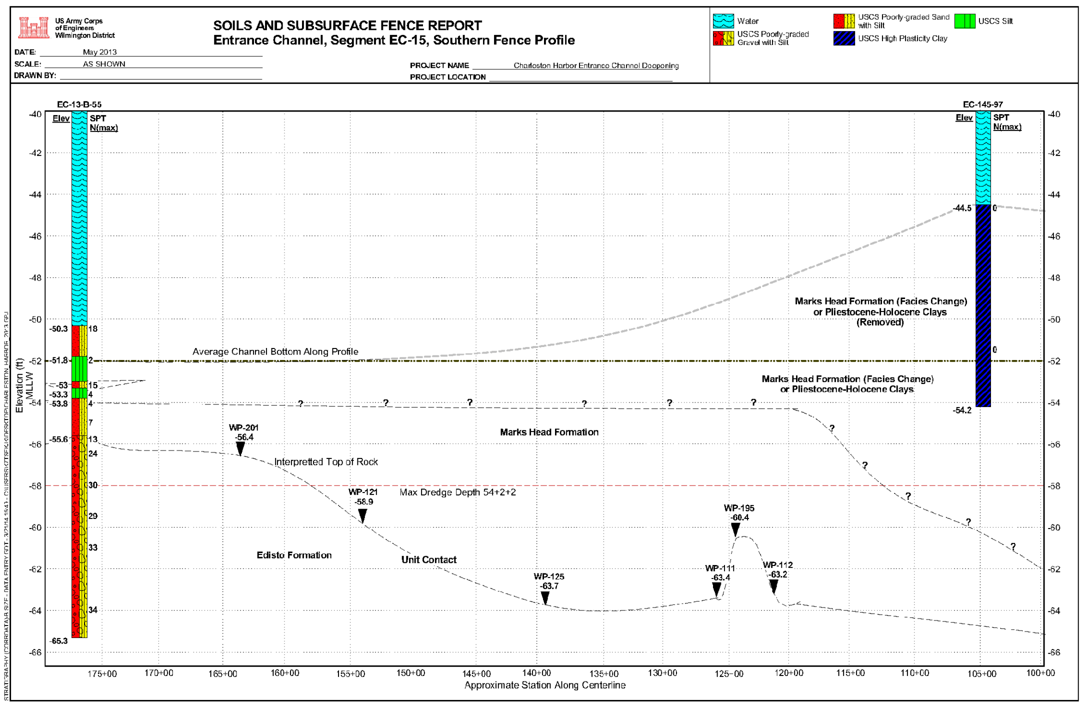


Figure B-64: Harbor Stratigraphy Entrance Channel, Segment EC-15







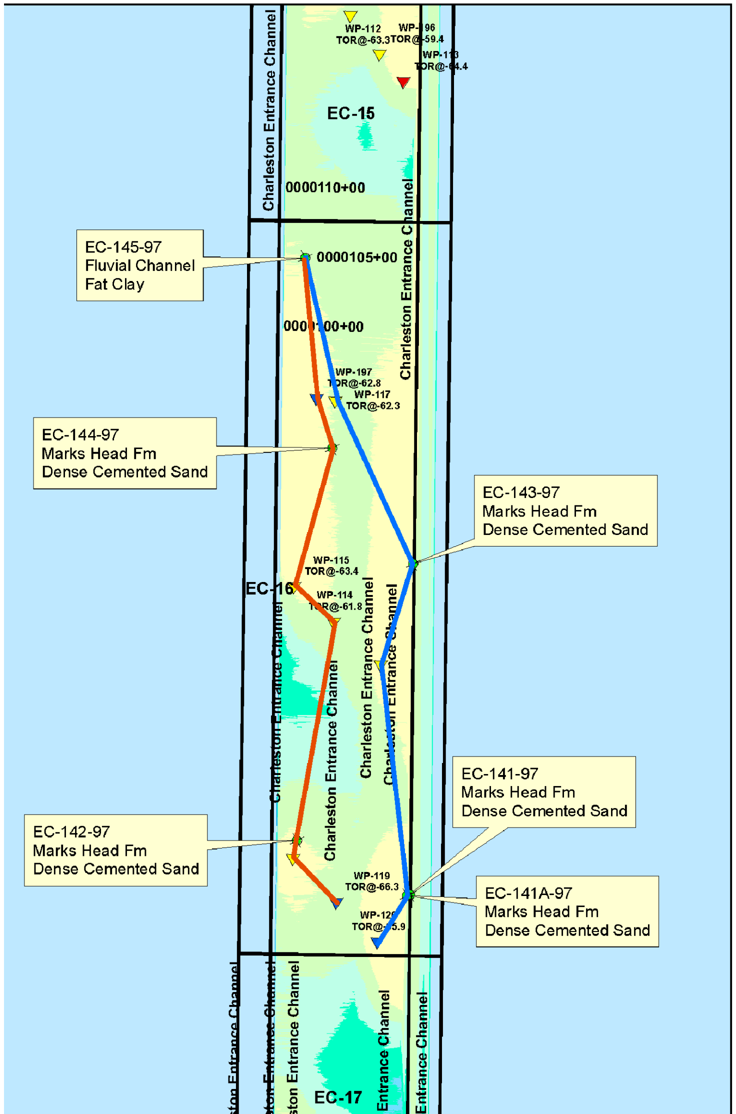


Figure B-65: Harbor Stratigraphy Entrance Channel, Segment EC-16

0 500 1,000 2,000 3,000 Feet

NOTE: Bathymetric color-contoured surface is based upon single-beam sonar condition survey, conducted by CESAC on 25JUN2013.

Legend

Borings
*2013 Boring Info Colored Pink
Rock Core
SPT
Vibracore

Northern Profile

Southern Profile

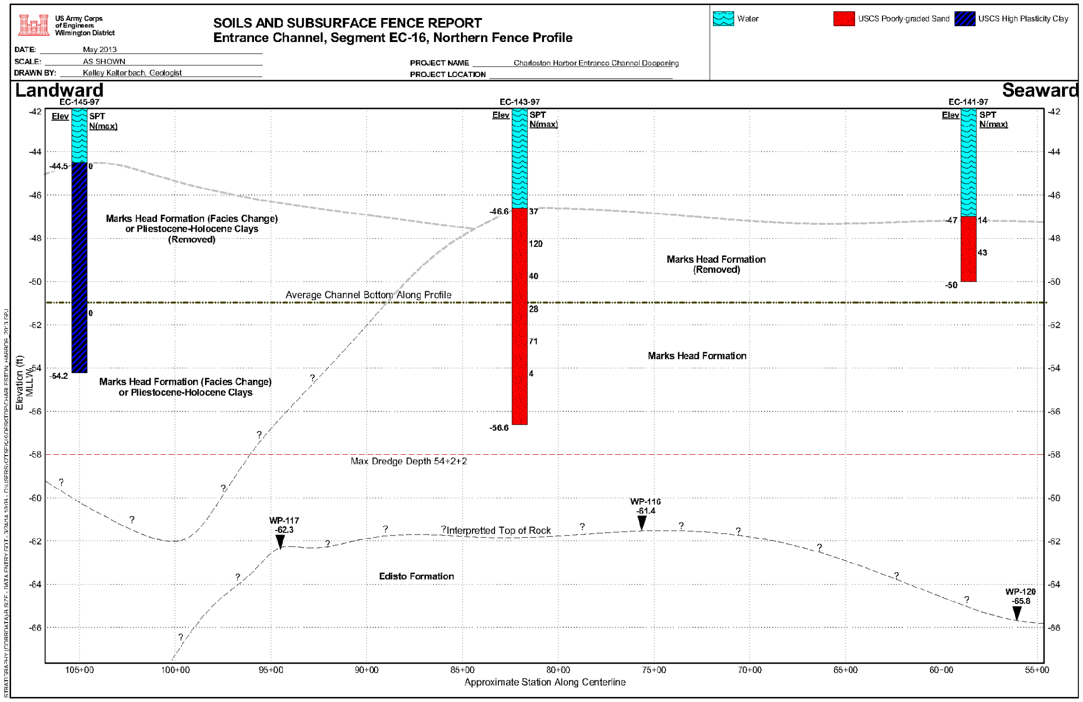
Channel Segments

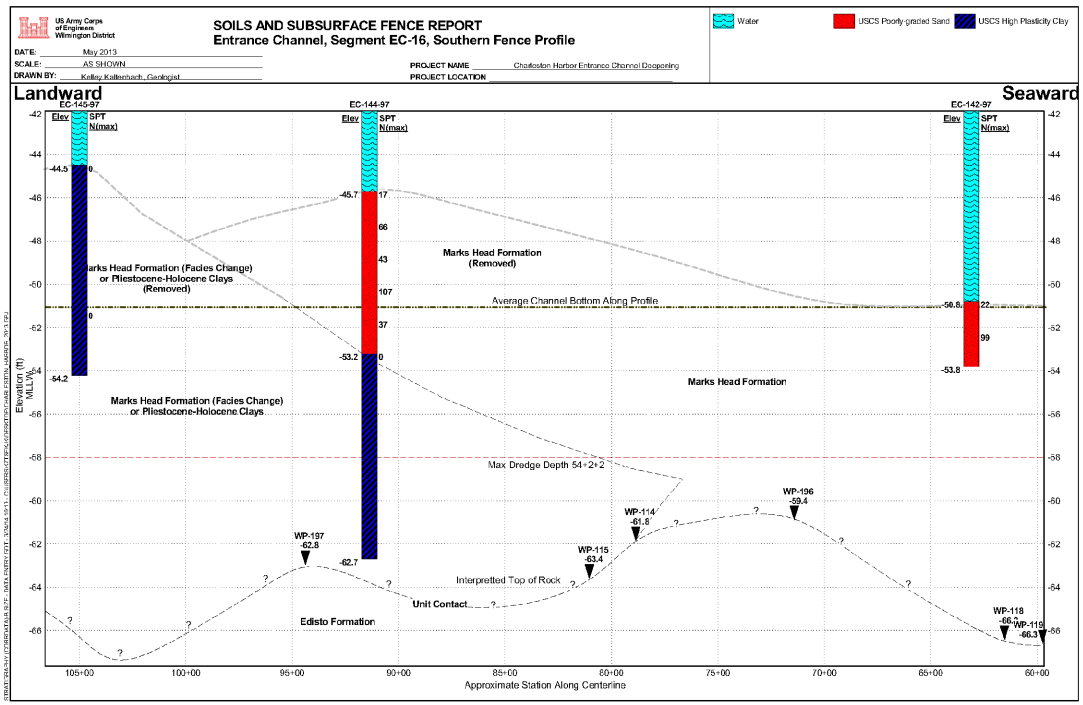
Athena 2013 Washprobes
Refusal within Dredge Prism
Below -56 MLLW
No Refusal Encounted
Poss Rock Encountered

Charleston Harbor Entrance Channel Color-Coded Bathymetry (MLLW)

25.2 - 30	44 - 46	56 - 58
30 - 34	46 - 48	58 - 60
34 - 38	48 - 52	60 - 65
38 - 40	52 - 54	65 - 70
40 - 44	54 - 56	70 - 75

Map Scale: 1:9,000





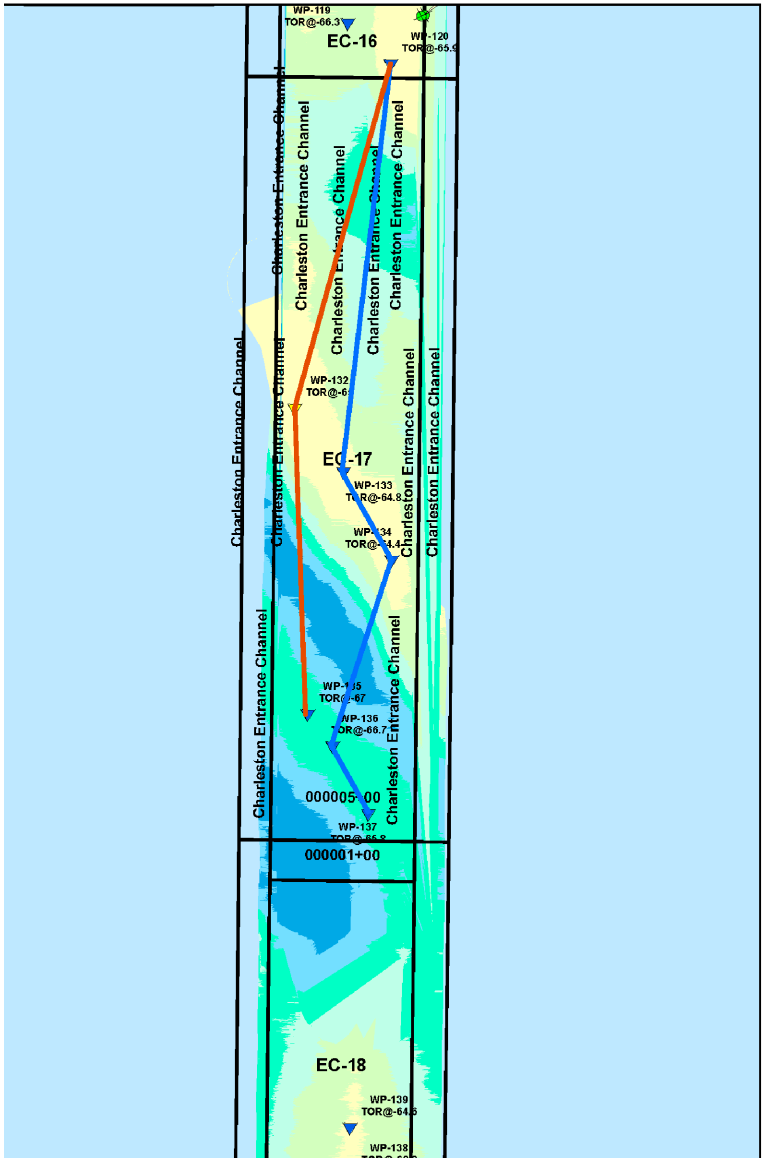


Figure B-66: Harbor Stratigraphy Entrance Channel, Segment EC-17

0 500 1,000 2,000 3,000 Feet

NOTE: Bathymetric color-contoured surface is based upon single-beam sonar condition survey, conducted by CESAC on 25JUN2013.

Legend

Borings
*2013 Boring Info Colored Pink
Rock Core
SPT
Vibracore

Northern Profile

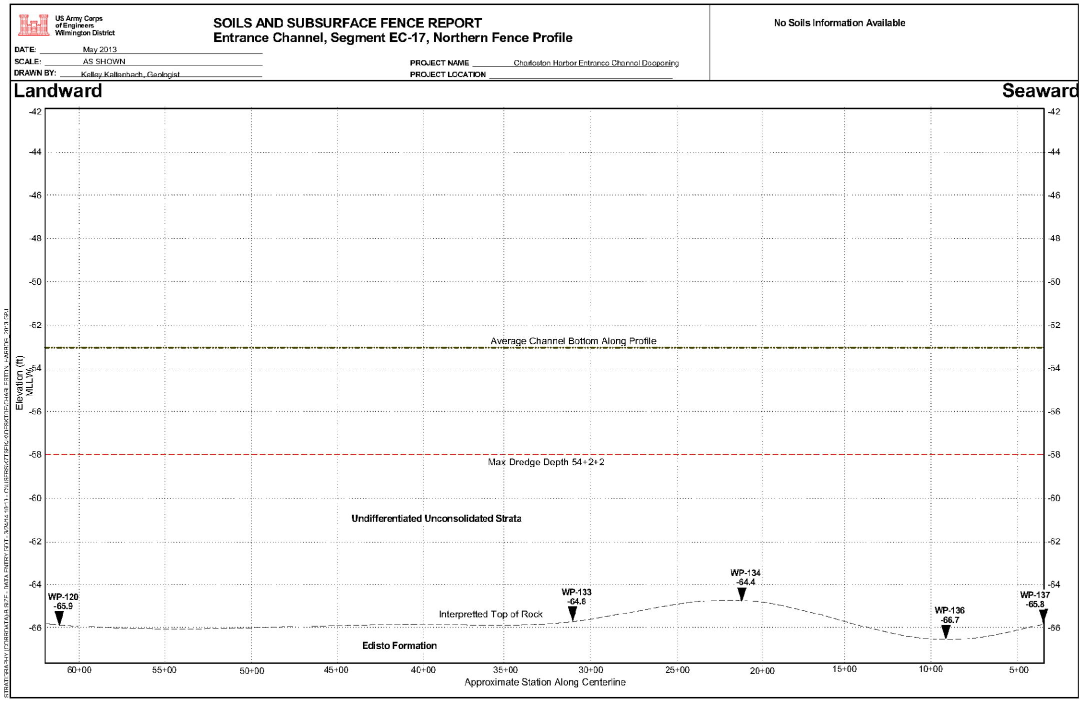
Southern Profile

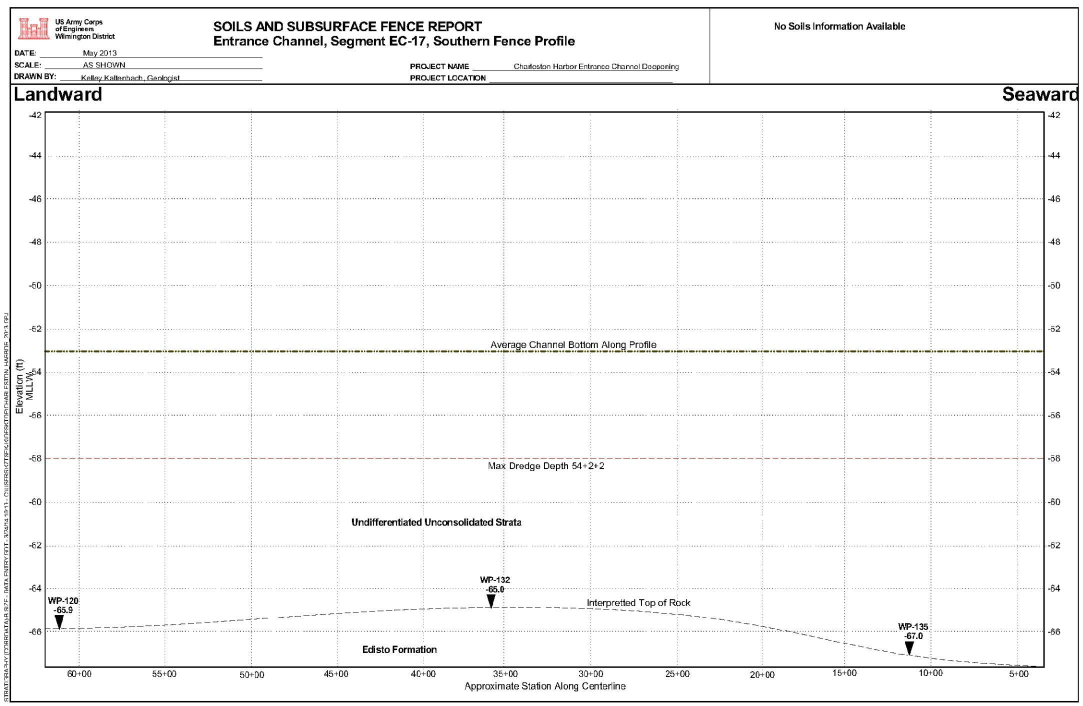
Channel Segments

Athena 2013 Washprobes
Refusal within Dredge Prism
Below -56 MLLW
No Refusal Encountered
Poss Rock Encountered

Charleston Harbor Entrance Channel Color-Coded Bathymetry (MLLW)
25.2 - 30
30 - 34
34 - 38
38 - 40
40 - 44
44 - 46
46 - 48
48 - 52
52 - 54
54 - 56
56 - 58
58 - 60
60 - 65
65 - 70
70 - 75

Map Scale: 1:9,000





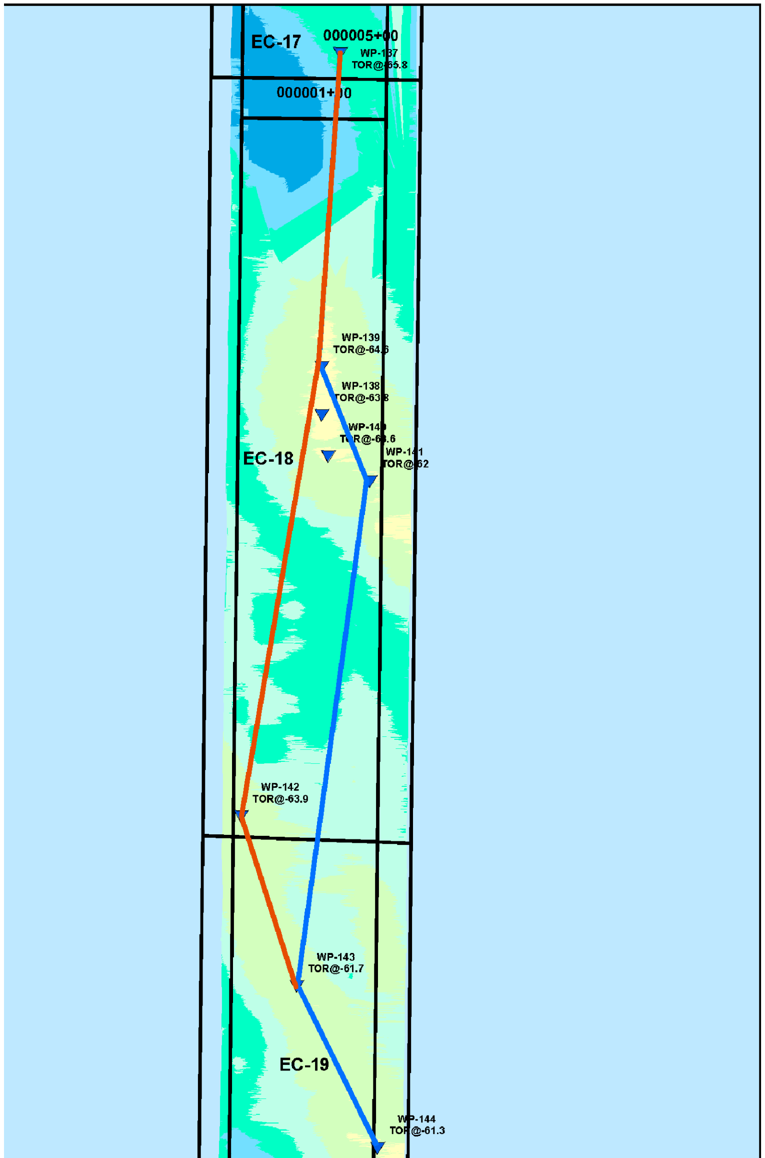


Figure B-67: Harbor Stratigraphy Entrance Channel, Segment EC-18

0 500 1,000 2,000 3,000 Feet

NOTE: Bathymetric color-contoured surface is based upon single-beam sonar condition survey, conducted by CESAC on 25JUN2013.

Legend

Borings
*2013 Boring Info Colored Pink
- Rock Core
- SPT
- Vibracore

Northern Profile

Southern Profile

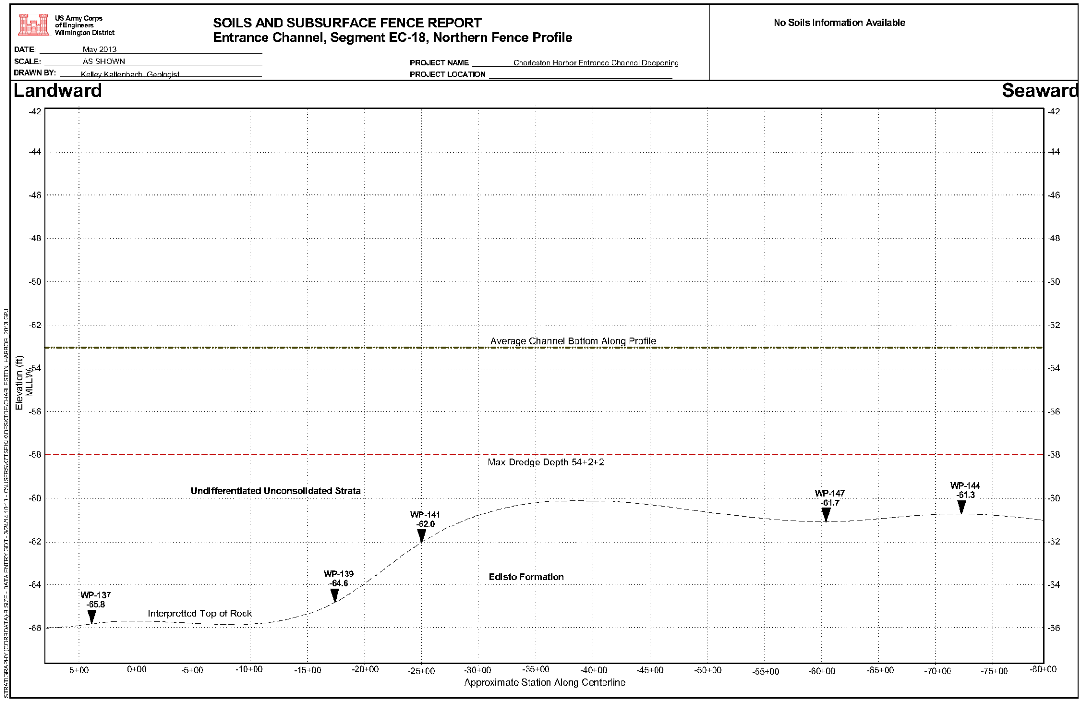
Channel Segments

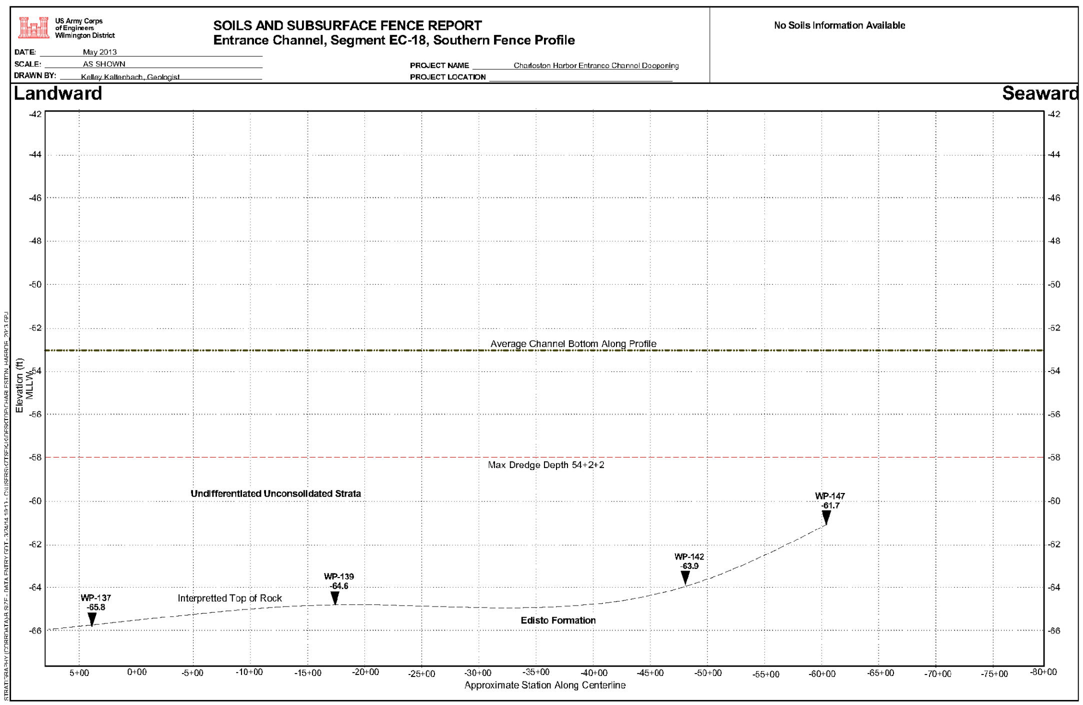
Athena 2013 Washprobes
- Refusal within Dredge Prism
- Below -56 MLLW
- No Refusal Encounted
- Poss Rock Encountered

Charleston Harbor Entrance Channel Color-Coded Bathymetry (MLLW)

25.2 - 30	44 - 46	56 - 58
30 - 34	46 - 48	58 - 60
34 - 38	48 - 52	60 - 65
38 - 40	52 - 54	65 - 70
40 - 44	54 - 56	70 - 75

Map Scale: 1:9,000





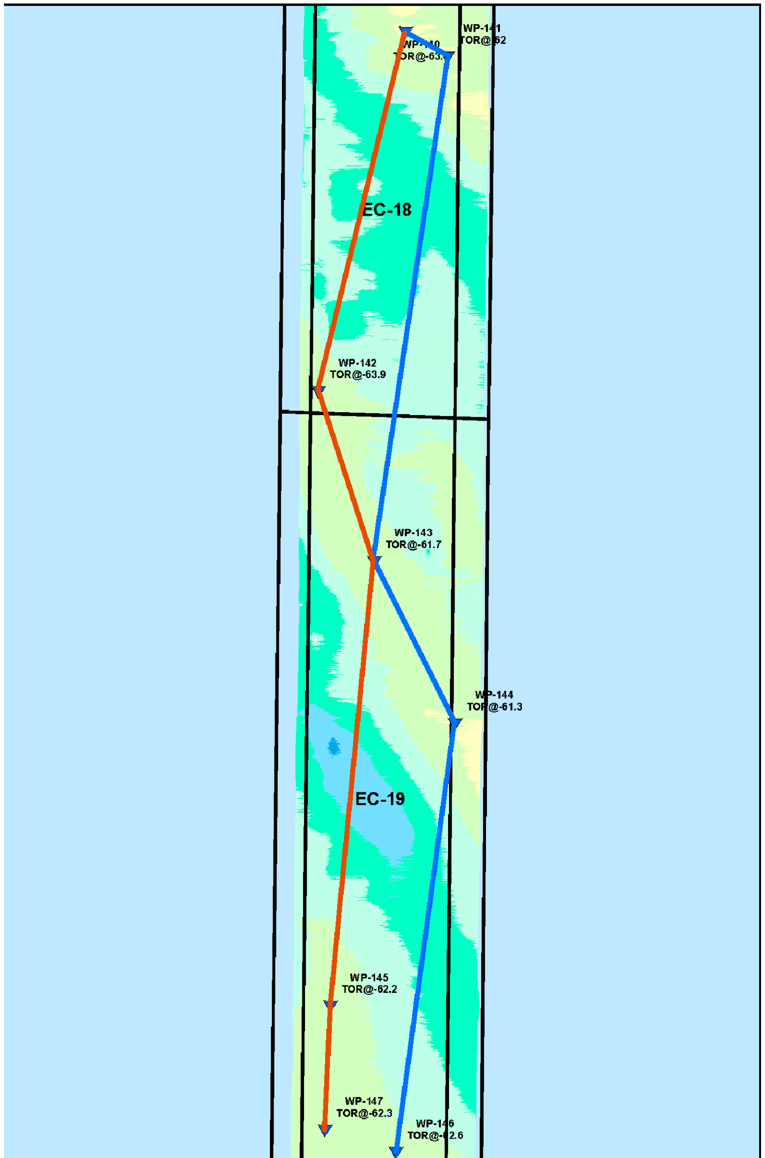
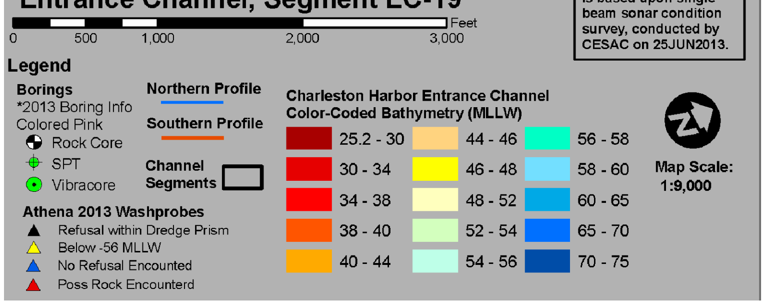
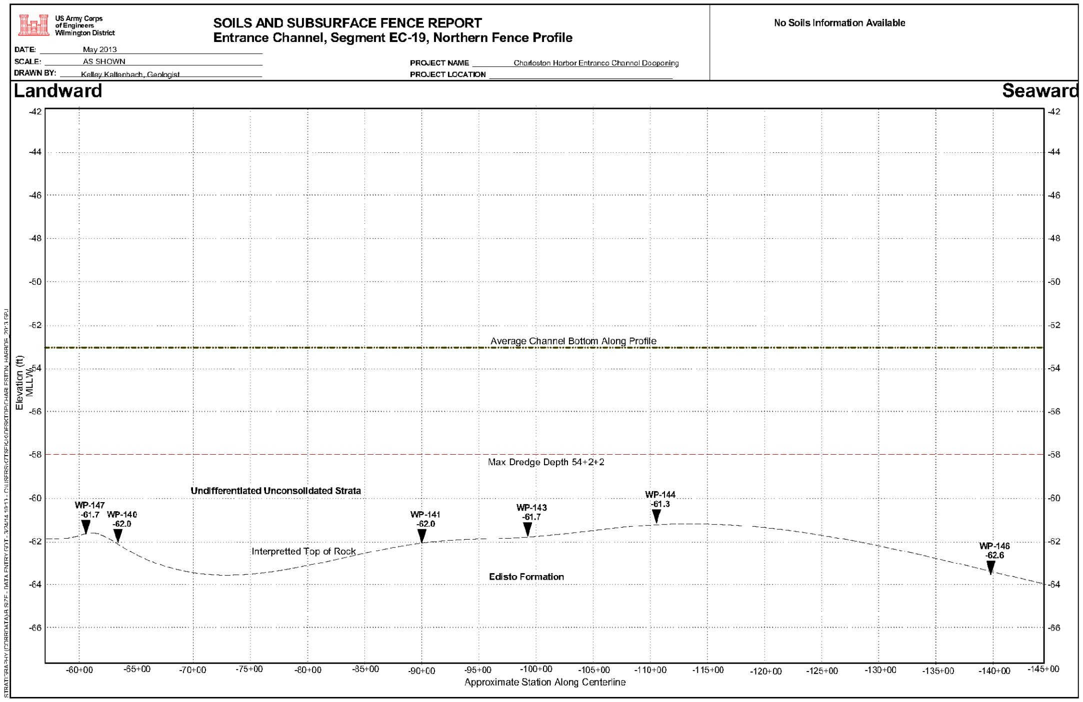
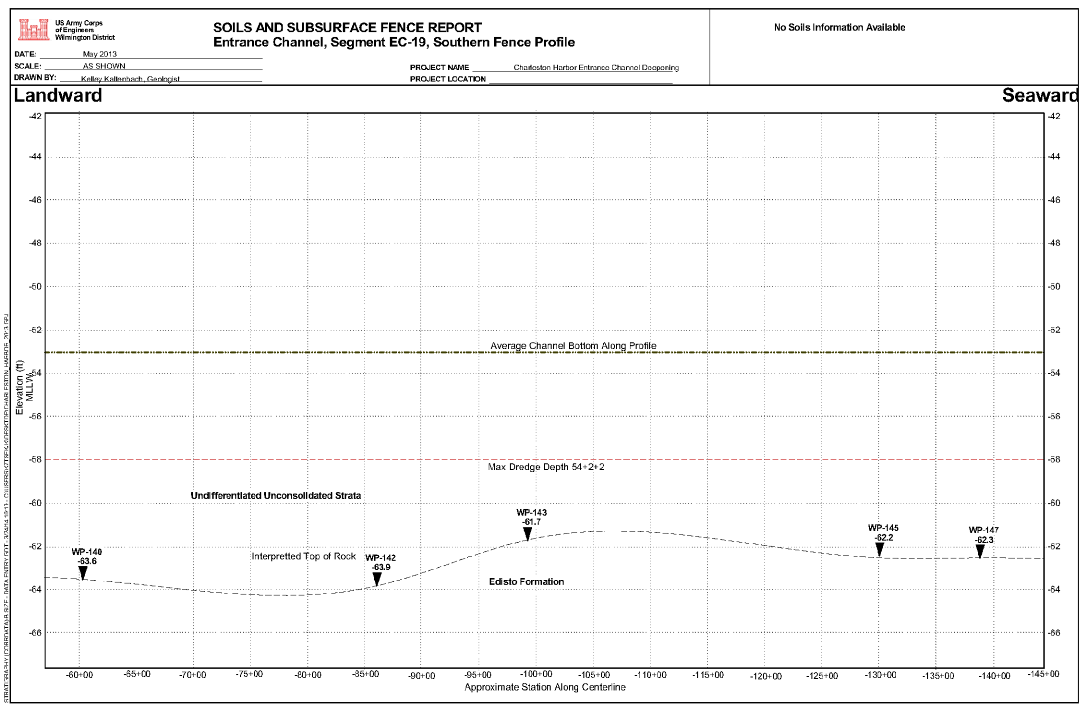


Figure B-68: Harbor Stratigraphy Entrance Channel, Segment EC-19

NOTE: Bathymetric color-contoured surface is based upon single-beam sonar condition survey, conducted by CESAC on 25JUN2013.







5.7.20 Stratigraphic Summary

A summary table that shows the predominant geologic materials that can be expected to be encountered if the channel is deepened to -58 feet MLLW is shown below. SPT N-values for fine-grained and granular material are listed for reference.

Table B-15. Entrance Channel Stratigraphic Summary

Figure	Reach	Predominant Material	SPT-N (fine-grained)	SPT-N (granular)
B-50	Entrance Channel, EC-1	Inorganic Silt, Clayey Sand	2 - 16	0 - 19
B-51	Entrance Channel, EC-2	Inorganic Silt, Clayey Sand	0 - 18	1 - 81
B-52	Entrance Channel, EC-3	Inorganic Silt, Fat Clay, Silty Sand	5 - 12	3 - 12
B-53	Entrance Channel, EC-4	Inorganic Silt, Silty Sand	7 - 12	5 - 14
B-54	Entrance Channel, EC-5	Silty Sand, Sand, Limestone, Silt	4 - 9	8 - 46
B-55	Entrance Channel, EC-6	Limestone, Clayey-Silty Sand, Sand	---	15 -40
B-56	Entrance Channel, EC-7	Limestone, Silty Sand, Sand, Silt	2 - 4	6 - 42
B-57	Entrance Channel, EC-8	Limestone, Silty-Clayey Sand, Sand	---	3 - 29
B-58	Entrance Channel, EC-9	Limestone, Fat Clay, Silty Sand	0 - 5	11 - 100
B-59	Entrance Channel, EC-10	Limestone, Silty Sand, Sand	---	2 - 91
B-60	Entrance Channel, EC-11	Limestone, Silty Sand, Sand	---	11 – 76
B-61	Entrance Channel, EC-12	Limestone, Silty Sand, Sand	---	18 – 74
B-62	Entrance Channel, EC-13	Limestone, Sand	---	12 - 36
B-63	Entrance Channel, EC-14	Sand, Gravel	---	12 - 30
B-64	Entrance Channel, EC-15	Sand, Gravel, Silt, Clay	0 - 4	7 – 30
B-65	Entrance Channel, EC-16	Fat Clay, Sand	0	22 - 99
B-66 to 68	Entrance Channel EC-17 to 19	No material data available	Assume < 2	Assume < 4

5.8 Mapping and Volume Estimates of Limestone within the Entrance Channel

5.8.1. Geologic Strip Map

The subsurface materials encountered during drilling vary laterally along the length of the entrance channel, as well as vertically. The lateral distribution of sediments roughly corresponds to the stratigraphic framework and geologic mapping of the Charleston area by Weems and Lemon (1993). A geologic strip map was initially developed using the 2013 boring data, because it was during the drilling operations in which the full extent of the Edisto Formation in the channel was recognized. The intact limestone rock cores can be correlated to previous investigations where the geologist characterized disarticulated limestone recovered from SPT drilling as a gravel or sand. The limestone is largely based upon a silty sand matrix with variable amounts of shell, which is consistent with previous workers descriptions. Given this correlation, the historical data was then re-analyzed and used refine the unit boundaries. A revised geologic strip map (Plate 12) was then developed that combines both 2013 and historical drilling data shows the lateral variation of geologic materials within the entrance channel.

Limestone bedrock belonging to the Edisto Formation occurs within channel segments EC-4 through EC-13 (see Plate 12). Drilling records (Attachment B-2) indicate that there are lesser amounts of limestone along the northern sides of channel segments EC-6 and EC-7. What may be interpreted as northerly trending paleofluvial channel system is incised into the limestone bedrock within EC-5, EC-6, and EC-7 (see Plate 4, Plate 12, Figures B-54 to B-56). The majority of the limestone is located within channel segments EC-5, EC-7 and EC-8 through EC-12 (see Figures B-54, B-56 and B-57 through B-61).

5.8.2. Area Dimensions

The estimated area and maximum thickness of limestone bedrock within the proposed dredging prism is provided in the table below. The thickness estimates include cemented granular soils such as limestone gravels; this material is interpreted to be top of limestone bedrock.

Table B-16. Maximum dimensions of rock per segment based drilling data.

Channel Segment	Area (sq. feet)	Max Thickness (feet)
EC-4	1,114,646	2.5
EC-5	4,145, 692	12.9
EC-6	2,188, 318	7.3
EC-7	3,028,295	6.6
EC-8	4,500, 286	10.0
EC-9	5,433,416	11.2
EC-10	5,560,563	6.6
EC-11	5,759,802	7.2
EC-12	5,756,055	8.4
EC-13	3,720,418	8.6

5.8.3. Revised Rock Volume Estimate

The results from the 2013 drilling program were used to revise the excavation rock volumes to facilitate better project cost estimation. The method used to calculate the new work rock volume requires that the geometries of the top of rock (TOR) and the proposed channel prism be subtracted from each other by 3-D vector analysis using Hypack, Microstation, or ArcGIS software.

Wilmington District, USACE created a composite TOR dataset that combined the historical drilling data with the washprobe and rock cores drilled in 2013. The dataset was formatted as an XYZ point data set where the easting and northing coordinates of the source borings represent the X and Y values accordingly, and the elevation of TOR represents the Z value. Each drilling record had to meet screening criteria before it was used order to build TOR point dataset. Entrance channel borings were visually scanned for descriptions that contained limestone, coquina, limestone gravel, calcareous sand, cemented sand, and shelly sand, which is recognized as an indicator of material belonging to the Edisto Formation. Once recognized, these borings were separated and a set of principles were applied to establish top of rock elevations for each data point;

- TOR = elevation of top of rock within borings
- TOR = elevation at which limestone gravel is first recognized in the boring
- TOR = Bathymetric surface in historical borings that contain calcareous soils and gravels that extend above the present (25JUN13) bathymetric condition survey.
- TOR = completion elevation in borings that lie within horizontal boundaries of the Edisto Formation, but may have been drilled within paleo-fluvial channels that are incised into the limestone bedrock.

These principles are conservative, because the natural TOR surface may be deeper or less well defined, but they were necessary in order to maintain the data density required to build the TOR surface. The TOR dataset (Attachment B-5) was then given to USACE-Charleston District for computational analysis. SAC personnel conducted several iterations of volume calculations using ArcGIS and Hypack software separately in order to assure quality control. The results of the volume calculations are presented in Table B-17. Revised estimates include volumes for rock to be removed that lies within the design template, rock lying above the design template[23], and the total amount to be removed. The majority of the rock lies within segments EC-4 to EC-13. The total volume of rock that is estimated to need removal for a -58 foot MLLW channel is 9,698,919 cubic yards. This estimate is 2-3 times greater than the original estimate of 3,476,646 cubic yards, but is considered more accurate because the geology of the channel is much better defined.

Table B-17. Revised volume estimates of limestone within the entrance channel.

Estimated Material Quantities Undifferentiated (CY)		PRE-2013 EXPLORATION-HISTORICAL RECORDS ONLY	% Type Material Within -58 MLLW Dredgiing Prism (Based Upon 1986-1999 Borings)				Initial 2012-2013 Estimate Rock Volume (CY)	POST- 2013 EXPLORATION DRILLING & TOP OF ROCK SURFACE DEVELOPMENT	2014 Revised Rock Volume Estimate (CY)		
	58'		% Unconsolidated	% Soft Rock	% Hard Rock	% Unknown			Total Rock Calculated (58')	Rock Above Condition	Rock Needing Removal (58')
Segment 1	569,596		76%	0%	0%	24%	0		0	0	0
Segment 2	435,529		58%	17%	5%	19%	98,720		0	0	0
Segment 3	625,978		59%	7%	0%	34%	44,713		0	0	0
Segment 4	737,540		35%	52%	0%	14%	380,117		1,482,956	238,272	**1,244,684**
Segment 5	729,419		46%	34%	11%	9%	329,509		1,167,207	9,809	**1,157,398**
Segment 6	652,831		52%	38%	0%	10%	249,584		863,488	10,370	**853,118**
Segment 7	573,134		62%	33%	0%	5%	187,686		972,260	65,274	**906,986**
Segment 8	507,662		54%	35%	6%	5%	208,271		878,613	57,003	**821,610**
Segment 9	476,307		38%	24%	34%	3%	279,830		1,074,904	202,113	**872,791**
Segment 10	550,547		30%	16%	47%	7%	347,359		1,175,070	167,258	**1,007,812**
Segment 11	517,333		17%	5%	73%	5%	405,458		1,013,277	63,134	**950,143**
Segment 12	450,290		18%	30%	52%	0%	368,809		1,355,248	186,918	**1,168,330**
Segment 13	430,406		17%	33%	50%	0%	358,671		741,992	25,945	**716,047**
Segment 14	287,713		0%	0%	0%	100%	0		0	0	0
Segment 15	289,292		0%	0%	0%	100%	0		0	0	0
Segment 16	367,736		35%	31%	28%	6%	217,918		0	0	0
Segment 17	188,858		0%	0%	0%	100%	0		0	0	0
Segment 18	118,868		0%	0%	0%	100%	0		0	0	0
Segment 19	147,116		0%	0%	0%	100%	0		0	0	0
Segment 20	108,614		0%	0%	0%	100%	0		0	0	0
Segment 21	2,470		0%	0%	0%	100%	0		0	0	0
Total QTY (CY)	**8,767,238**						**3,476,646**		10,725,015	1,026,096	**9,698,919**

5.9 Summary of Lab Testing

5.9.1. Soil Test Results

Attachment B-3 contains the material gradation data and lab results. A summary of these results is provided in Table B-18. The majority of the materials submitted for testing were granular in nature, while only 15 samples were fine-grained. The laboratory visual classification of granular materials tended to be finer grained than the field visual classification. This difference is likely due to a number of factors; field biases in the observation of the material, subsequent desiccation of granular soils, mechanical breaking of intergranular cemented bonds during test preparation and sieving, etc.

[23] "Rock Above Condition" refers to bedrock that lies within the current channel that GLDD did not remove during the last deepening in 1999. This rock lies somewhere between the 2-foot allowable over depth and advanced maintenance dredging prisms used by SAC Navigation/Operations. Material volumes were considered separately due to Hypack/ArcGIS data processing.

Table B-18. Summary of 2013 Entrance Channel Material Properties from USACE-EMU.

				D6913 % Passing								
Lab Number	Hole Number	Sample Number	Depth (ft) MLLW	No.4 %	No 200 %	D4318 Atterberg Limits LL	PL	PI	D2216 MC%	Color	Class Symbol	D2487 Unified Soil Classification System
K2/3289	EC-13-B-1	1	52.0 to 53.5	96.8	52.8	44	31	13	23.0	Very Dark Grayish Brown	ML	Sandy Inorganic Silt Low LL (ML), with a trace of gravel.
K2/3292	EC-13-B-1	4	56.5 to 58.0	100.0	22.9	41	36	5	40.6	Dark Olive Gray	SM	Silty Sand (SM).
K2/3297	EC-13-B-2	3	55.9 to 57.4	100.0	53.3	50	45	5	33.3	Very Dark Grayish Brown	MH	Sandy Inorganic Silt High LL (MH).
K2/3301	EC-13-B-3	2	57.3 to 58.8	100.0	52.4	47	41	6	30.0	Dark Olive Gray	ML	Sandy Inorganic Silt Low LL (ML).
K2/3303	EC-13-B-3	4	60.3 to 61.8	99.4	27.6	---	---	---	36.4	Dark Olive Gray	SM	Silty Sand (SM).
K2/3306	EC-13-B-4	2	55.5 to 57.0	100.0	15.4	---	---	---	35.9	Very Dark Gray	SM	Silty Sand (SM).
K2/3308	EC-13-B-4	4	59.2 to 60.7	100.0	24.9	---	---	---	37.1	Very Dark Gray	SM	Silty Sand (SM).
K2/3310	EC-13-B-4	6	62.2 to 63.7	100.0	51.8	---	---	---	35.6	Dark Olive Gray	ML	(Visual) Sandy Inorganic Silt Low LL (ML).
K2/3316	EC-13-B-5	2	52.9 to 54.4	100.0	33.4	---	---	---	48.3	Black	SM	Silty Sand (SM).
K2/3318	EC-13-B-5	4	55.9 to 57.4	100.0	19.0	---	---	---	39.0	Black	SM	Silty Sand (SM).
K2/3320	EC-13-B-5	6	58.9 to 60.4	99.7	19.8	---	---	---	36.9	Black	SM	Silty Sand (SM).
K2/3322	EC-13-B-6	2	52.3 to 53.8	100.0	29.6	63	43	20	38.1	Black	SM-H	Silty Sand High LL (SM-H).
K2/3323	EC-13-B-6	3	54.3 to 55.8	100.0	33.2	75	58	17	48.1	Black	SM-H	Silty Sand High LL (SM-H).
K2/3325	EC-13-B-6	5	57.3 to 58.8	100.0	23.7	---	---	---	42.0	Black	SM	Silty Sand (SM).
K2/3330	EC-13-B-7	2	53.4 to 54.9	100.0	30.4	---	---	---	37.7	Black	SM	Silty Sand (SM).
K2/3332	EC-13-B-7	4	56.7 to 58.2	100.0	21.5	---	---	---	36.4	Black	SM	Silty Sand (SM).
K2/3335	EC-13-B-7	7	61.8 to 63.3	100.0	23.0	---	---	---	36.6	Very Dark Gray	SM	Silty Sand (SM).
K2/3338	EC-13-B-8	2	54.2 to 55.7	100.0	55.4	---	---	---	41.3	Black	MH	(Visual) Sandy Inorganic Silt High LL (MH).
K2/3340	EC-13-B-8	4	57.2 to 58.7	100.0	30.6	64	49	15	33.0	Black	SM-H	Silty Sand High LL (SM-H).
K2/3342	EC-13-B-8	6	60.2 to 61.7	100.0	21.0	---	---	---	41.4	Black	SM	Silty Sand (SM).
K2/3345	EC-13-B-9	2	52.9 to 54.4	100.0	79.4	96	52	44	51.7	Very Dark Gray	MH	Inorganic Silt High LL (MH), with some sand.
K2/3347	EC-13-B-9	4	55.9 to 57.4	100.0	73.9	---	---	---	53.8	Black	MH	(Visual) Inorganic Silt High LL (MH), with some sand.
K2/3349	EC-13-B-9	6	58.9 to 60.4	100.0	58.5	---	---	---	54.1	Black	MH	(Visual) Sandy Inorganic Silt High LL (MH).
K2/3351	EC-13-B-9	8	61.9 to 63.4	100.0	39.4	94	63	31	46.8	Black	SM-H	Silty Sand High LL (SM-H).
K2/3355	EC-13-B-10	3B	52.3 to 52.6	58.7	19.4	---	---	---	25.3	Olive	SM	Gravelly Silty Sand (SM).
K2/3356	EC-13-B-10	4	53.1 to 54.6	100.0	87.7	132	40	92	52.1	Very Dark Gray	CH	Fat Clay (CH), with a little sand.
K2/3358	EC-13-B-10	6	56.6 to 58.1	95.5	72.0	119	68	51	59.7	Dark Olive Gray	MH	Inorganic Silt High LL (MH), with some sand and a trace of gravel.
K2/3361	EC-13-B-10	9	61.7 to 63.2	100.0	40.0	---	---	---	45.1	Black	SM	Silty Sand (SM).
K2/3364	EC-13-B-11	2	55.8 to 57.3	86.7	28.3	---	---	---	30.2	Olive	SM	Silty Sand (SM), with a little gravel.
K2/3365	EC-13-B-11	3	57.3 to 58.8	78.3	25.6	---	---	---	35.5	Olive	SM	Silty Sand (SM), with some gravel.
K2/3366	EC-13-B-11	4	58.8 to 60.3	100.0	59.6	74	33	41	49.7	Very Dark Gray	CH	Sandy Fat Clay (CH).
K2/3369	EC-13-B-12	2	53.8 to 55.3	97.4	21.4	---	---	---	34.1	Olive Gray	SM	Silty Sand (SM), with a trace of gravel.
K2/3371	EC-13-B-12	4	56.8 to 58.3	90.4	23.2	---	---	---	30.4	Olive Gray	SM	Silty Sand (SM), with a trace of gravel.
K2/3373	EC-13-B-12	6	59.8 to 61.3	99.5	40.6	---	---	---	40.8	Olive Gray	SC	(Visual) Clayey Sand (SC).
K2/3374	EC-13-B-12	7	61.3 to 62.8	100.0	89.9	100	32	68	54.4	Very Dark Gray	CH	Fat Clay (CH), with a little sand.
K2/3376	EC-13-B-13	2	51.6 to 53.1	98.5	19.6	---	---	---	24.5	Gray & Light Gray	SM	Silty Sand (SM), with a trace of gravel.
K2/3380	EC-13-B-13	6	57.7 to 59.2	98.3	15.8	---	---	---	31.2	Gray	SM	Silty Sand (SM), with a trace of gravel.
K2/3381	EC-13-B-13	7	59.2 to 60.7	91.2	14.7	---	---	---	31.6	Gray	SM	Silty Sand (SM), with a trace of gravel.

5.9.2. Rock Testing Results

Attachment B-4 contains the laboratory rock strength data sheets. A summary of this testing is provided in Table B-19. A total of 65 unconfined compressive strength tests were run once on each of the submitted core samples. The minimum and maximum UC strengths encountered were 73.7 psi and 415.8 psi respectively. The average UC strength is 162.5 psi. A total of 80 Brazilian splitting tensile strength tests were run on the samples submitted, in addition to duplicates cut from untested UC-sample trimmings. The minimum and maximum tensile strength encountered were 0.7 psi and 136 psi. The average rock tensile strength is 37.1 psi, which is 23% or roughly a quarter of the average UC strength.

Table B-19. Summary of 2013 Entrance Channel Rock Strength Testing from USACE-EMU

Lab Number	Boring #	Sample #	Elevation Interval	Test	Diameter	UCS (psi)	STS-A (psi)	STS-B (psi)	STS-C (psi)
K2/3203	EC-13-B-28	1	53.4-53.7	STS	HQ		11.0		
3204	EC-13-B-28	2	54.1-54.6	UCS	HQ	88.8			
3205	EC-13-B-28	3	57.0-57.5	UCS	HQ	97.6			
3206	EC-13-B-28	4	57.7-58.1	UCS	HQ	95.2			
3207	EC-13-B-28	5	58.8-59.3	UCS	HQ	56.7			
3208	EC-13-B-28	6	59.5-59.8	STS	HQ		19.1	19.9	18.5
3209	EC-13-B-32	1	55.3-55.6	STS	HQ		64.7	76.0	61.5
3210	EC-13-B-32	2	56.0-56.5	UCS	HQ	189.4			
3211	EC-13-B-32	3	58.1-58.6	UCS	HQ	249.7			
3212	EC-13-B-33	1	53.1-53.5	UCS	HQ	350.9			
3213	EC-13-B-33	2	55.0-55.4	UCS	HQ	237.8			
3214	EC-13-B-33	3	56.0-56.4	STS	HQ		37.9		
3215	EC-13-B-33	4	58.5-58.9	UCS	HQ	322.1			
3216	EC-13-B-34	1	56.4-56.8	STS	HQ		14.8		
3217	EC-13-B-34	2	57.7-58.2	UCS	HQ	124.7			
3218	EC-13-B-34	3	59.7-60.2	UCS	HQ	194.6			
3219	EC-13-B-35	1	53.7-54.1	STS	HQ		2.5	10.5	
3220	EC-13-B-35	2	55.0-55.5	UCS	HQ	195.0			
3221	EC-13-B-35	3	59.0-59.5	UCS	HQ	231.0			
3222	EC-13-B-36	1	54.3-54.8	UCS	HQ	183.9			
3223	EC-13-B-36	2	56.7-57.2	UCS	HQ	145.4			
3224	EC-13-B-37	1	53.6-53.9	STS	HQ		15.7		
3225	EC-13-B-37	2	55.3-55.8	STS	HQ		24.0	11.2	
3226	EC-13-B-37	3	59.2-59.7	UCS	HQ	174.5			
3227	EC-13-B-38	1	56.2-56.7	UCS	HQ	33.3			
3228	EC-13-B-38	2	57.7-58.0	STS	HQ		34.1	26.5	11.8
3229	EC-13-B-38	3	59.0-59.5	UCS	HQ	100.7			
3230	EC-13-B-39	1	54.2-54.7	UCS	PQ	176.5			
3231	EC-13-B-39	2	55.2-55.7	STS	PQ		59.0	89.8	50.4
3232	EC-13-B-39	3	57.2-57.7	UCS	PQ	248.9			
3233	EC-13-B-39	4	58.7-59.3	UCS	PQ	253.3			
3234	EC-13-B-39	5	59.3-59.8	STS	PQ		31.3	64.5	37.7
3235	EC-13-B-40	1	53.7-54.3	UCS	PQ	295.5			
3236	EC-13-B-40	2	55.8-56.3	UCS	PQ	292.9			
3237	EC-13-B-40	3	56.7-57.7	STS	PQ		70.8	56.7	66.5
3238	EC-13-B-40	4	58.7-59.3	UCS	PQ	232.1			
3239	EC-13-B-41	1	53.6-54.1	UCS	PQ	186.0			
3240	EC-13-B-41	2	55.9-56.4	UCS	PQ	226.3			
3241	EC-13-B-41	3	57.4-57.8	STS	PQ		36.6	77.1	86.2
3242	EC-13-B-41	4	58.6-59.0	STS	HQ		40.9	74.3	33.9
3243	EC-13-B-41	5	59.5-60.0	UCS	HQ	273.7			
3244	EC-13-B-42	1	53.0-53.5	UCS	PQ	223.3			
3245	EC-13-B-42	2	54.6-55.1	UCS	PQ	195.2			
3246	EC-13-B-42	3	55.8-56.1	STS	PQ		31.8	22.6	
3247	EC-13-B-42	4	57.9-58.4	UCS	PQ	200.1			
3248	EC-13-B-42	5	59.3-59.6	STS	PQ		60.4	70.1	82.5

Lab Number	Boring #	Sample #	Elevation Interval	Test	Diameter	UCS (psi)	STS-A (psi)	STS-B (psi)	STS-C (psi)
3249	EC-13-B-43	1	54.0-54.5	UCS	PQ	369.2			
3250	EC-13-B-43	2	55.4-55.8	STS	PQ		63.2	56.3	36.6
3251	EC-13-B-43	3	56.6-57.1	UCS	PQ	415.8			
3252	EC-13-B-43	4	58.3-58.8	UCS	PQ	219.3			
3253	EC-13-B-43	5	59.3-59.7	STS	PQ		136.0	113.5	112.4
3254	EC-13-B-44	1	56.8-57.3	UCS	PQ	114.6			
3255	EC-13-B-44	2	58.4-58.8	STS	PQ		40.7	17.7	21.3
3256	EC-13-B-44	3	59.4-59.9	UCS	PQ	158.7			
3257	EC-13-B-45	1	53.7-54.2	UCS	PQ	227.4			
3258	EC-13-B-45	2	55.0-55.5	STS	PQ		31.7	26.8	32.1
3259	EC-13-B-45	3	55.8-56.3	UCS	PQ	200.5			
3260	EC-13-B-45	4	57.8-58.3	UCS	PQ	191.4			
3261	EC-13-B-45	5	59.5-60.0	STS	PQ		24.4	52.2	
3262	EC-13-B-46	1	57.5-58.0	UCS	PQ	138.4			
3263	EC-13-B-46	2	59.0-59.5	STS	PQ		2.8	42.8	56.2
3264	EC-13-B-46	3	59.9-60.4	UCS	PQ	170.5			
3265	EC-13-B-47	1	56.1-56.7	UCS	PQ	130.5			
3266	EC-13-B-47	2	57.2-57.7	STS	PQ		22.2		
3267	EC-13-B-47	3	58.5-59.0	UCS	PQ	152.3			
3268	EC-13-B-48	1	52.7-53.2	UCS	PQ	98.4			
3269	EC-13-B-48	2	52.9-53.4	UCS	PQ	204.9			
3270	EC-13-B-48	3	57.1-57.6	STS	PQ		13.6		
3271	EC-13-B-48	4	57.7-58.2	UCS	PQ	89.1			
3272	EC-13-B-48	5	59.7-60.2	UCS	PQ	142.4			
3273	EC-13-B-48	6	58.7-59.2	STS	PQ		38.9	30.3	55.6
3274	EC-13-B-49	1	53.1-53.7	UCS	PQ	84.8			
3275	EC-13-B-49	2	55.7-56.2	UCS	PQ	88.1			
3276	EC-13-B-49	3	56.6-56.9	STS	PQ		8.4		
3277	EC-13-B-49	4	58.4-58.9	UCS	PQ	0.0			
3278	EC-13-B-50	1	51.6-52.1	UCS	HQ	115.3			
3279	EC-13-B-50	2	53.2-53.6	UCS	HQ	73.7			
3280	EC-13-B-50	3	58.3-58.6	STS	HQ		22.8	26.5	18.1
3281	EC-13-B-51	1	51.5-51.9	UCS	PQ	76.4			
3282	EC-13-B-51	2	52.9-53.4	UCS	PQ	77.0			
3283	EC-13-B-51	3	54.2-54.7	STS	PQ		19.0		
3284	EC-13-B-51	4	56.0-56.6	UCS	HQ	95.3			
3285	EC-13-B-51	5	58.4-58.7	STS	HQ		20.8		
3286	EC-13-B-52	1	57.9-58.4	UCS	PQ	107.2			
3287	EC-13-B-52	2	59.8-60.3	UCS	PQ	101.0			
3288	EC-13-B-52	3	57.0-57.4	STS	PQ		13.4	18.7	17.6
3502	EC-13-B-18	1	53.9-54.4	UCS	HQ	139.8			
3503	EC-13-B-18	2	55.0-55.3	STS	HQ		11.5	6.9	10.4
3504	EC-13-B-18	3	57.3-57.8	UCS	HQ	139.1			
3505	EC-13-B-18	4	58.6-58.9	STS	HQ		26.6		
3506	EC-13-B-18	5	59.4-59.9	UCS	HQ	122.4			
3507	EC-13-B-20	1	57.2-53.2	UCS	HQ	209.9			
3508	EC-13-B-20	2	55.6-56.0	STS	HQ		5.1		
3509	EC-13-B-20	3	57.2-57.6	STS	HQ		10.6	4.3	
3510	EC-13-B-20	4	58.7-59.2	UCS	HQ	154.7			
3511	EC-13-B-21	1	53.5-54.0	UCS	HQ	120.3			
3512	EC-13-B-21	2	54.9-55.2	STS	HQ		18.2		
3513	EC-13-B-21	3	56.0-56.5	UCS	HQ	150.8			
3514	EC-13-B-21	4	57.9-58.4	UCS	HQ	158.0			
3515	EC-13-B-21	5	59.1-59.4	STS	HQ		29.1	12.1	0.7
3516	EC-13-B-24	1	56.0-56.5	UCS	PQ	77.4			
3517	EC-13-B-24	2	57.5-58.0	UCS	PQ	79.7			
3518	EC-13-B-24	3	58.5-58.8	STS	PQ		21.8		
3519	EC-13-B-24	4	59.5-59.8	STS	PQ		14.5		
3287	EC-13-B-52	2	59.8-60.3	UCS	PQ	101.0			
3288	EC-13-B-52	3	57.0-57.4	STS	PQ		13.4	18.7	17.6
3502	EC-13-B-18	1	53.9-54.4	UCS	HQ	139.8			
3503	EC-13-B-18	2	55.0-55.3	STS	HQ		11.5	6.9	10.4
3504	EC-13-B-18	3	57.3-57.8	UCS	HQ	139.1			
3505	EC-13-B-18	4	58.6-58.9	STS	HQ		26.6		

Lab Number	Boring #	Sample #	Elevation Interval	Test	Diameter	UCS (psi)	STS-A (psi)	STS-B (psi)	STS-C (psi)
3506	EC-13-B-18	5	59.4-59.9	UCS	HQ	122.4			
3507	EC-13-B-20	1	57.2-53.2	UCS	HQ	209.9			
3508	EC-13-B-20	2	55.6-56.0	STS	HQ		5.1		
3509	EC-13-B-20	3	57.2-57.6	STS	HQ		10.6	4.3	
3510	EC-13-B-20	4	58.7-59.2	UCS	HQ	154.7			
3511	EC-13-B-21	1	53.5-54.0	UCS	HQ	120.3			
3512	EC-13-B-21	2	54.9-55.2	STS	HQ		18.2		
3513	EC-13-B-21	3	56.0-56.5	UCS	HQ	150.8			
3514	EC-13-B-21	4	57.9-58.4	UCS	HQ	158.0			
3515	EC-13-B-21	5	59.1-59.4	STS	HQ		29.1	12.1	0.7
3516	EC-13-B-24	1	56.0-56.5	UCS	PQ	77.4			
3517	EC-13-B-24	2	57.5-58.0	UCS	PQ	79.7			
3518	EC-13-B-24	3	58.5-58.8	STS	PQ		21.8		
3519	EC-13-B-24	4	59.5-59.8	STS	PQ		14.5		

5.10 Rock Dredgeability

5.10.1. Parameters used to Determine Rock Dredgeability by Rock Cutter-Head

USACE-Wilmington District used the following rock strength parameters to determine rock dredgeability; unconfined compressive strength, splitting tensile strength, percent core recovery, rock quality designation, and the thickness of bedrock. Of these parameters, it has been the collective experience[24] within Wilmington District that the unconfined compressive strength of the rock plays the greatest role in the determination of its dredgeability.

The unconfined compressive strength of rock is one of the most widely regarded indicators of rock dredgeability (USACE, 1983; Hignett, 1984; Smith, 1987, 1994; Bieniawski, 1989; Vervoort and DeWitt, 1997). These workers have indicated through their individual fields of expertise that the UCS is the best indicator of material dredgeability. Hignett (1984) reported that the maximum unconfined compressive strength that rock cutter head dredges could effectively remove ranged from 3625 psi to 4351 psi, even though their individual components were rated for much stronger rock. These figures were given for 1970’s to 1980’s era dredges, which have probably been upgraded in capacity in the 30 years since the publication. The other parameters become increasingly important when strong rock is encountered and the dredging contractor must alter his plan of work in order to utilize natural planes of weakness within the rock for economic removal. Above 4351 psi, the rock must be blasted to allow removal (Hignett, 1984).

In the case of the Wilmington Harbor Anchorage Basin, the average unconfined compressive strength of the in-situ rock was 548 psi, with a strength range from 301 psi to 1364 psi. The Anchorage Basin was assessed by the Wilmington District to be dredgeable, but there were initial concerns to rock dredgeability in areas that had rock strengths in excess of 500 psi and thicknesses greater than 4-feet (Figure B-69). Great Lakes Dock and Dredging mobilized the *D/B Texas* to the site in December 2012 and removed all of the rock in the Anchorage Basin without the need for blasting. The rock mass in the area of concern was removed easily without incident.

[24] Based on rock dredging experience from Wilmington Harbor, which has much harder limestone than Charleston Harbor. Specific rock dredging projects include the Baldhead Shoals Re-alighnment and Anchorage Basin Deepening. Coastal southeastern NC has similar geology as Charleston, SC, but the bedrock is much better cemented. Wilmington Harbor could be considered a more extreme case in terms of rock strength and cementation, than Charleston Harbor.

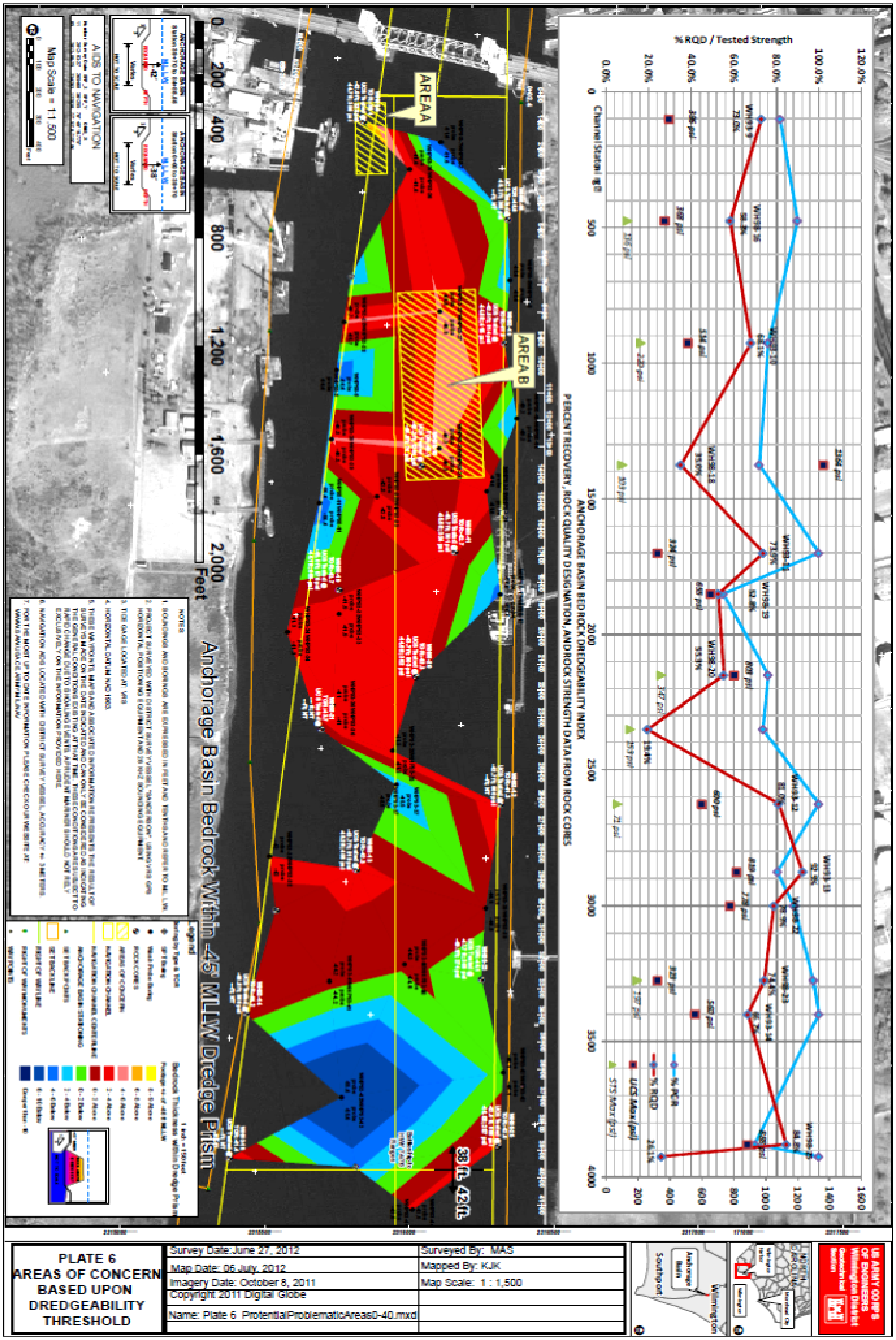


Figure B-52. Wilmington Harbor Anchorage Basin problematic areas > 500 psi & > 4-feet thick.

5.10.2. Strength of Materials within the Entrance Channel

The strength of the material sampled during the 2013 drilling program was tabulated in Excel, and plotted against the existing maps, as illustrated in Plates 13 and 14. The maximum N-blow count from all SPT sampling (1988 to 2013) is plotted against channel stationing for segments EC-1 through EC-16. SPT N-values for the recent drilling are plotted in red, while the historical SPT values are plotted in dark blue. The maximum unconfined compressive strength (UCS) of limestone samples taken within the dredging prism (< -58 MLLW) are plotted as red point data, alongside historical UCS test data from USACE (black) and GLDD (gray).

The Cooper Formation floors much of channel segments EC-1 into most of EC-4 (Figure B-54 through Figure B-56). This fine-grained, silty-clayey material is medium stiff to very stiff based upon SPT N-values that range from 4 to 19. No limestone was encountered within channel segments EC-1 through EC-3. The materials in these segments are not cemented and should be considered low-strength. Historical data indicates that the limestone may occur as thin, discontinuous beds within EC-4.

Transitional sand or paleofluvial material floor the northern side of channel segments EC-5, EC-6, EC-7 and a small portion of EC-8 (Figure B-57 through Figure B-60). These materials have variable amounts of cementation and compaction, which appear to have a wide variation of relative density. The graph of SPT N-values in Plate 14 indicates that the density of these materials range from loose (N = 4) to dense (N = 40). The higher densities are considered indicative the limestone that is shown to lie along the southern bank of these channel segments. Borings along the northern bank that have relatively high blow count values may have intercepted zones of deeply indurated limestone, or coarse-grained detrital material that was shed off the limestone subcroppings along the southern bank.

Subsurface data indicates that the density and relative strength of material increases from EC-5 to EC-6. Rock sampled from these sections is no more than 210 psi in strength. A small erosional window of Cooper Marl is denoted in EC-7, which roughly corresponds to a drop in SPT N-values below 5. An increase in SPT N-values to N=20 indicates the presence of denser granular material which was encountered in boring EC-13-B-22. Seaward of boring EC-13-B-22, more weakly cemented limestone (98 psi) crops out from the seafloor and increases in quantity within segment EC-8. Channel segment EC-9 is floored by weakly cemented limestone that ranges in strength from 114 to 500 psi, based upon GLDD claims data and the 2013 lab testing.

The strength of the limestone present in channel segments EC-10 through EC-13 (Figure B-59 to Figure B-62) is less than 450 psi, based upon the results of the 2013 lab testing. When compared to the GLDD UCS data, most of the rock strengths are much weaker. The highest rock strength values are within the GLDD dataset, notably UC strengths of 994 psi and 1670 psi. However, as discussed in section 2.4.2, these values do not represent the overall strength of the rock mass, but rather the strength of isolated well-silicified, discontinuous strata, and should be considered data outliers. Therefore the strength range of the limestone bedrock is generally constrained to 450 psi or less.

Based upon the low strength of the rock within the entrance channel, and the ease by which stronger rock was removed from Wilmington Harbor’s Anchorage Basin by rock cutter head alone, there should be no need for blasting in Charleston Harbor. The rock that is present should be easily removable by a modern rock cutter head dredge.

5.10.3. Seismic Vibration

Seismic vibration generated from rock cutter-head dredging should pose no risk to existing structures within Charleston. There are two lines of reasoning for this;

1. The location where rock dredging will occur is distant from any structure. Any seismic waves generated will be sufficiently attenuated below established peak particle velocity (PPV) damage thresholds. For reference, rock dredging conducted in Wilmington Harbor, was located 1-2 miles from the downtown historical district, and cutter-head vibration never exceeded the established PPV threshold.

2. Foundation soils in Charleston have already been subjected to relatively high PPV's from previous large magnitude earthquakes. Foundation structures may have already settled as a result of liquefaction of the underlying non-cohesive soils (where present). Furthermore, multiple earthquake events may have induced settlement of foundation soils, effectively buffering any settlement effects (however unlikely) from the seismic waves generated from the cutter-head.

5.11 Conclusions

- The limestone previously encountered by Great Lakes Docks and Dredging belongs to the Edisto Formation and is much more widespread than initially anticipated.

- Volume estimates using TOR modeling and the proposed channel template (-58 MLLW) indicate that the volume of rock that will need to be removed is 9,698,919 cubic yards. This estimate is 2-3 times greater than the original estimate of 3,476,646 cubic yards, but is considered more accurate because the geology of the channel is much better defined.

- Overall, the unconfined compressive strength of tested samples indicates that the limestone is very weak and soft. Low unconfined compressive strength bedrock is very conducive to removal by rock cutter head dredging.

- Previous investigations underestimated the extent of the rock in the entrance channel because of its low unconfined compressive strength. The rock, when sampled using the SPT method, returned disarticulated sand and gravel, which suggested that it was unconsolidated. This contributed to a change of condition claim by GLDD in 1999.

- Based upon the available drilling logs and lab data, and using conservative engineering-geology judgment, the limestone bedrock should not need blasting for removal.

- The need for vibration monitoring is not anticipated for this project.

- Additional drilling and/or laboratory testing for the entrance channel should not be required during PED due to the sampling coverage provided in 2013.

VI. CLOUTER CREEK

6.1 Introduction

Clouter Creek Disposal Area (DA) is a diked upland area that is used to contain material dredged from the Cooper River for navigational purposes. It is located east of North Charleston, on the east bank of the Cooper River. The east side of Clouter Creek DA is bordered by Clouter Creek, while the north, south, and west sides are bordered by the Cooper River. Totaling roughly 1,475 acres, Clouter Creek DA is divided into four "cells", South Cell, Middle Cell, Highway Cell, and North Cell. The approximate acreages are as follows:

Table B-20.

Clouter Creek DA Area	
South Cell	415 Acres
Middle Cell	410 Acres
Highway Cell	460 Acres
North Cell	190 Acres

The portion of the Cooper River dredged material placed into Clouter Creek Disposal Area consists of the upper harbor, from the Daniel Island Reach to the Ordnance Reach. The northern third of Clouter Creek DA is owned by the South Carolina State Ports Authority (SCSPA), and the southern two-thirds are owned by the U.S. Army Corps of Engineers. The Federal Government enjoys a perpetual easement on the state owned portion.

6.2 Fifty Year Future Life Cycle

6.2.1 Current Dredging Volume

The upper harbor reaches are dredged on a bi-annual basis (every 18-24 months). The yearly dredge material average that is placed into Clouter Creek DA is 837,216 cubic yards. Authorized third party users also place dredged material into Clouter Creek DA on a yearly basis, with an average annual volume of 448,749 cubic yards. The total average annual dredged material disposal amount that is placed in Clouter Creek Disposal Area is almost 1.3 million cubic yards.

6.2.2 New Work

New work is divided into two areas: upper harbor individual reaches and wideners. The current authorized dredging depth in the upper harbor is 45-feet, plus 2 to 4-feet of advanced maintenance and an additional 2-feet allowable overdepth, for a total depth of 49-feet. The exception to this are areas of high shoaling[25] which have additional allowance for maintenance dredging. Minimum new work depth is 47-feet, plus 2-feet advanced maintenance and 2-feet allowable overdepth for a total depth of 51'. Maximum new work depth is 52-feet, plus 2-feet advanced maintenance and 2-feet allowable overdepth for a total of 56-feet, with additional

[25] High shoaling areas in Lower Wando, Lower Town Creek, Ordnance Reaches, Ordnance Turning Basin, and Wando Turning Basin are required to have 45' depth with 4' of authorized advanced maintenance dredging and an additional 2' allowable overdepth. Drum Island Reach is required to have 45', plus 6' of authorized advanced maintenance, and an additional 2' allowable overdepth.

allowance for high shoaling areas. Wideners are to be dredged to the same depth as the channel segments. Maximum new work depth is 52’, plus 2’ advanced maintenance and 2’ allowable overdepth for a total of 56’. The new work volume of dredged material ranges from 373,481 cubic yards to almost 6 million cubic yards. See Table B-21 for individual quantities. A critical design issue for the proposed dike raises to accommodate current and new work dredging volume is settlement and stability.

Table B-21.

New Work Volume (cy)*										
Reaches	**47'**	**48'**	**49'**	**50'**	**51'**	**52'**	**53'**	**54'**	**55'**	**56'**
Daniel Island Reach				125,375	300,709	519,440	773,667	1,041,015	1,314,719	1,592,690
Daniel Island Bend				15,962	37,045	74,551				
Clouter Creek Reach				96,155	232,407	389,959				
Navy Yard Reach				81,661	211,072	358,816				
N Charleston Reach				33,372	109,877	225,645				
Fiblin Creek Reach				23,387	69,348	156,072				
Port Terminal Reach				27,374	78,918	160,376				
Ordinance Reach						30,989	72,331	118,091		
Ordinance Reach Turning Basin						56,845	116,170	176,617		
Wideners (Maximum Option)	**47'**	**48'**	**49'**	**50'**	**51'**	**52'**	**53'**	**54'**	**55'**	**56'**
Daniel Island Reach			386,121	411,412	451,556	478,874	499,692	527,341	548,115	576,062
Clouter Creek Reach	77,292	97,588	119,837	143,280	167,650	193,191				
N Charleston Reach	163,555	189,374	216,743	245,331	276,119	307,048				
Fiblin Creek Reach	117,449	140,583	165,494	192,131	220,283	249,348				
Fiblin-Port Terminal Intersection	15,185	17,998	21,052	24,357	27,924	31,692				
Ordinance Reach Turning Basin					1,193,600	1,253,007	1,311,876	1,372,696		
Total	373,481	445,543	909,247	1,419,797	3,376,508	4,485,851	4,920,434	5,262,767	5,676,935	5,982,853

6.2.3 Proposed Dike Raise to Accommodate Current and New Work Volumes[26].

A 50 year dredged volume was calculated, as well as the new work volume for the upper harbor deepening to 56’. The total capacity shortfall at Clouter Creek DA is approximately 64 million cubic yards (mcy). With a total acreage of 1475 at Clouter Creek DA, a raise of 26.9’ would be required to place all the material for the 50-year dredge volume. This excludes the extra capacity that is gained from utilizing the material from inside the DA to complete the dike raises. Numerous dike raises will be required to gain a 50-year capacity for Clouter Creek DA, with a final top elevation of approximately 50’ (NAVD88). Each incremental raise will be approximately 5’ in height. Levee raises design and analyses were conducted in accordance with EM 1110-2-5072, Confined Disposal of Dredged Material.

[26] Data table from SAC, Operations Branch, circa September 2012. As per Caleb Brewer, maintenance dredging material must also be accounted for in disposal to Clouter Creek. He specifically mentions that “…Going back and adding in the areas that are not being studied for deepening, but material still goes to Clouter Creek is where the 837,216 cubic yards per year comes from. The original yearly average of 837,216 cy yr is the correct average for all reaches in which material is disposed of in Clouter Creek”.

High strength geotextile would be placed with every dike raise to ensure the Factor of Safety (F.S.)[27] remains above 1.3. Each raise will be analyzed for slope stability and settlement prior to the designing of the raise. For the North, Highway, Middle, and South cells, each raise would also include a step-in, placing the next dike raise to the inside toe of the previous raise, as well as a fifty foot berm placed to the inside of the cell. The cross dike between the North and Highway cells, Highway and Middle cells, and Middle and South cells would be raised along the centerline.

6.3 Subsurface Investigation

Historical data was researched and data deficiencies were identified in order to locate areas on Clouter Creek DA which require further subsurface data. In October 2012, Cone Penetration Testing (CPT) was performed in those areas where data was deemed insufficient. Standard Penetration Testing (SPT) was performed in November and December 2013 at the previous CPT locations.

6.3.1 Field Methods

6.3.1.1 Cone Penetration Testing (CPT). In December 2012, the U.S. Army Corps of Engineers, Savannah District, performed cone penetration testing (CPT) on Clouter Creek Disposal Area. The CPT is also a widely accepted test method of *in situ* testing of foundation soils (ASTM D 5778) and provides a relatively inexpensive and rapid means for determining subsurface conditions. An instrumented conical shaped probe (60° cone tip, 10 centimeters in diameter, with the friction sleeve area 150 centimeters in diameter) is pushed into a soil deposit at a controlled rate of 2 cm/sec at each location to the termination depth. Depth of penetration is measured by an optical encoder, and is verified by manually measuring the depth of penetration and comparing the result to the final sounding depth measured by the encoder. The tip of the cone was instrumented to measure tip resistance (q_c) using strain gauges, while the attached sleeve was instrumented to measure friction (f_s) as the cone was advanced. The cone was also equipped with a pore pressure transducer to measure induced pore pressure or seismic sheer wave velocities (u_2) at discrete depth locations. Induced pore pressure is the excess pore water pressure generated by the probe displacing saturated soil. Low permeability soils will generate relatively high induced pore pressures, while high permeability soils will generate relatively low induced pore pressures. High permeability soils will generally show induced pore pressures that closely mirror hydrostatic pressures (u_0). The tip resistance, sleeve friction, and pore pressure were used to develop a profile of correlated soil type with depth. Output quantities for both sleeve friction and tip resistance are simultaneously recorded in units of tons per square foot per foot of depth. CPT testing provides a detailed record of cone resistance which is useful for evaluation of site stratigraphy. The use of the friction sleeve and pore-water pressure element is used to estimate soil classification and engineering properties of soils.

CPT testing was performed on 16 predetermined transects along the perimeter of all 4 cells of Clouter Creek Disposal Area (Figure B-70 and Figure B-71). Each transect consisted of 5 boring locations. These locations were: inside and outside embankment toe, inside and outside slope,

[27] **Factor of safety** (**F.S.**) is a term describing the structural capacity of a system beyond the expected loads or actual loads. F.S. describes how much stronger the system is than it needs to be for an intended load. Safety factors are calculated using detailed analysis because comprehensive testing is impractical; however, the structure's ability to carry the load must be determined to a reasonable accuracy.

and the crest. Of the 80 proposed CPT locations, only 67 were completed due to inaccessibility of the slope or toe locations. Several transects had steep outer slopes that dropped off to the marsh. In the instances where there was inadequate space to obtain all 5 testing locations, as many locations were tested as possible, allowing for the maximum collection of data.

Upon completion, all CPT borings were backfilled with bentonite grout. All CPT locations were recorded using a Trimble GeoXH GPS unit. Elevation data was acquired via LIDAR data provided by U.S. Army Corps of Engineer, Charleston District.

Figure B-70. Northern transect locations for Clouter Creek Disposal Area.

Figure B-71. Southern transect locations for Clouter Creek Disposal Area.

6.3.1.2 Standard Penetration Testing (SPT). In November and December, 2013, the U.S. Army Corps of Engineers, Savannah District, performed Standard Penetration Testing (SPT) on Clouter Creek DA. The test provides an indication of the relative density of granular soils, such as sand and gravel. Soil strength parameters derived from the test are generally considered approximate, but they are deemed acceptable given the widespread use of the method and it's relatively low cost. Correlation between the blow-count (N-value) and soil strength properties tends to be greater in sandy soils than in clayey soils. Despite this, the test method is used extensively to quantify soil properties for geotechnical engineering design.

SPT testing involves driving a standard thin-walled, 24-inch long, 2-inch OD/1-3/8-inch ID, splitspoon sampler a total depth of 18-inches into undisturbed soil. The driving energy for is imparted to the sampler (and length of drill rod) from the blows of a 140-lb hammer free-falling 30-inches. The number of blows to drive the sampler in three 6-inch increments is recorded. The first 6-inches of penetration is considered to be the seating drive. The sum of the number of blows required for the second and third 6-inches of penetration is termed the "standard penetration resistance" or the "N-value". The blows are applied and counted for each of the 6-inches until 18-inches of penetration is achieved. The test is terminated if: a total of 50- blows have been applied during any one of the three 6-inch increments, a total of 100-blows have been applied, or there is no observable advance in the sampler during the application of 10 successive blows of the hammer.

SPT testing was performed on eighteen predetermined locations along the perimeter of all 4 cells of Clouter Creek Disposal Area (Figure B-72). Of the proposed thirty-two SPT locations, only eighteen were completed due to timeline and funding constraints. Thirteen of the SPT holes were located where CPT testing was previously performed in 2012. The remaining five SPT holes were located at new locations around Clouter Creek DA.

SPT testing was performed in accordance with ASTM D1586, as well as ER-1110-1-1807. Each SPT boring was advanced by using a mud rotary auger with cleanout to the top of the next sample. Each boring began at the ground surface and was advanced in drive increments of 1.5-feet to -73.5 ft NAVD88. The first SPT was taken at a depth of 2-feet and then on 5-foot centers to the bottom of the hole. After each sample was taken, the splitspoon sampler was washed to prevent cross contamination with the next sample. An inspector from SAW was on site during the drilling operations to visually classify the soils and record the SPT blow counts at each 18-inch drive. The splitspoon samples were sealed in jars and taken to the SAD laboratory at the end of the sampling effort. A total of 270 splitspoon samples were collected from the SPT endeavor.

SPT holes were backfilled with grout by inserting PVC tremie pipe to the terminal depth. The tremie pipe was then filled with bentonite grout weighing approximately 100 lbs/ft^3 and then retracted, keeping the pipe topped off with grout until all sections were brought to the surface. All SPT sampling locations and elevations were recorded using a Trimble VRS GPS unit.

6.3.1.3 Undisturbed Sampling. At selected SPT boring locations, an adjacent “sister” UD test boring was advanced within 10’ horizontally from the SPT boring location for the purpose of collecting undisturbed samples. The undisturbed samples were labeled SPT-13-CC-X UD-x where “x” represents the corresponding SPT numbering and undisturbed sample number. The depth interval, date, and time were also identified for each sample. The undisturbed sample depths were determined at discretion of the SAW inspector, based on CPT data, as well as field classification results of soils at certain SPT locations.

Undisturbed sampling was performed in accordance with ASTM D1587. The thin-walled sampler tubes have an outside diameter of 3-inches and a total length of 30-inches. The undisturbed hole was advanced to the desired elevation using a mud rotary auger. The thin-walled samplers were then pushed for a penetration of 28-inches. After a thirty minute wait, the thin-walled sampler tube was removed from the boring, the recovery was measured, and the ends were sealed with wax and plastic caps. The tubes were labeled for orientation (top, bottom) and identification prior to being transported to the laboratory. Eighteen undisturbed samples were obtained, with some holes having two undisturbed samples taken and others having one undisturbed sample taken.

Figure B-72. SPT Boring locations for 2013 Clouter Creek Subsurface Investigation.

6.3.2 Laboratory Methods

6.3.2.1 ASTM D2216. Laboratory Determination of Water Content of Soil and Rock Mass. This test method covers the laboratory determination of the water (moisture) content by mass of soil where the reduction in mass by drying is due to the loss of water. For many materials, water content is one of the most significant index properties used in establishing a correlation between soil behavior and its index properties. The water content soil is used in expressing the phase relationships of air, water, and solids in a given volume of material. In fine-grained (cohesive) soils, the consistency of a given soil type depends on its water content. The water content of a soil, along with its liquid and plastic limit is used to express its relative consistency or liquidity index.

6.3.2.2 ASTM D2435. One-Dimensional Consolidation Properties of Soils Using Incremental Loading. This test method determines the magnitude and rate of consolidation of soil when restrained laterally and drained axially while subjected to incrementally applied controlled-stress loading. This test method is most commonly performed on undisturbed samples of fine grained soils naturally deposited in water. The data from the consolidation test are used to estimate the magnitude and rate of both differential and total settlement of earthen fill. Estimates of this type are of key importance in the design of engineered structures and the evaluation of their performance.

6.3.2.3 ASTM D2488. Description and Identification of Soils. This test method is used to identify soils based on visual examination and manual tests. Using visual examination and simple manual tests, soils can be identified using the classification group symbols and names. The descriptive information can be used to describe a soil to aid in the evaluation of its significant properties for engineering use.

6.3.2.4 ASTM D2850. Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils. This test method covers determination of the strength and stress-strain relationships of a cylindrical specimen of undisturbed cohesive soil. Specimens are subjected to a confining fluid pressure within a confined chamber. No drainage of the specimen is permitted during the test. The specimen is sheared in compression at a constant rate of axial deformation, without drainage. The compressive strength of a soil is determined in terms of the total stress; therefore, the material strength depends on the pressure developed in the pore fluid during loading. Fluid flow is not permitted from or into the soil specimen as the load is applied; therefore, the resulting pore pressure and strength differs from that developed in the case where drainage can occur.

6.3.2.5 ASTM D4318. Liquid Limits, Plastic Limits, and Plasticity Index of Soils. This test method is used to characterize the fine-grained fractions of soils. The liquid limit, plastic limit, and plasticity index of soils are also used with other soil properties to correlate with engineering behavior such as compressibility, hydraulic conductivity (permeability), compatibility, and sheer strength.

6.3.2.6 ASTM D4767. Consolidated Undrained Triaxial Compression Test for Cohesive Soils. This test method covers the determination of strength and stress-strain relationships of a cylindrical specimen of an undisturbed saturated cohesive soil. Specimens are isotropically consolidated and sheared in compression without drainage at a constant rate of axial deformation. The shear characteristics are measured under undrained conditions and are applicable to field conditions where soils that have been fully consolidated under one set of stresses are subjected to a change in stress without time for further consolidation to take place, and the field stress conditions are similar to those in the test method. The shear strength determined from the test is used in embankment stability analysis, earth pressure calculations, and foundation design.

6.3.2.7 ASTM D6913. Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis. This test method is used to determine the particle-size distribution (gradation) of a soil sample. A representative specimen is obtained from the sample after oven-drying. The specimen is sieved in its entirety, using a single sieve-set sieving. After the dry weight of the total sample was obtained, the sample was soaked in a dispersing agent. Once the samples had dispersed they were washed over a No. 200 sieve. The samples washed over the No. 200 sieve were then oven dried again and the dry weight after the No. 200 wash was recorded. If the sample weights indicated that over half of the material had passed the No. 200 sieve then no further testing was performed. However, if more than half of the sample was retained on the No. 200 sieve then the remaining portion of the sample was subjected to full sieve analysis after drying.

6.4 Settlement and Stability

6.4.1 Seepage Analysis

6.4.1.1 SEEP/W. Steady-state seepage analysis was performed using GeoStudio's SEEP/W, a two dimensional finite element modeling program. All analysis was conducted in accordance with EM 1110-2-5027, Confinced Disposal of Dredged Material. The phreatic surface and pore-pressure distribution was modeled for each dredging cycle after every raise for the fifty year life of the dike (Figure B-73). Levee cross sections were developed using subsurface data from the Cone Penetrometer Testing (CPT) data generated from the 2012 subsurface investigation and the 2013 as-built drawings supplied by the Charleston District from the 2012 LIDAR[28] topographic survey, then converted to finite element meshes. Hydraulic conductivity functions were defined, boundary conditions were applied, and seepage conditions were predicted for various dredging water elevations.

6.4.1.2 Seepage Analysis Assumptions and Input Parameters. For the preliminary designs, the dike profiles were determined from the 2013 Clouter Creek Disposal Area cross sections. These cross sections were developed from the 2012 LIDAR topographic survey conducted by the Charleston District (SAC). Three cross sections were constructed: North cell at N389698.4, E2325489, Highway Cell at N386298.5, E2325713, and Middle Cell at N382730.5, E2323906 (Figure 4). After a site visit to Clouter Creek DA, it was discovered that the existing data did not match current conditions at the Middle Cell, and that analysis was terminated. The North Cell was assumed to be a "typical" section of Clouter Creek DA, and the Highway Cell was modeled at the known failure area at that cross section. Both were modeled with dike raises to elevation 50'. A 3H:1V outside slope (riverside) and a 2H:1V inside slope (landside) was modeled for each raise, with high strength geotextile being placed at the ground level of each raise. The crest width is sixteen feet wide for each raise, and a fifty foot berm approximately three to four feet high is placed to the inside of the dike for stability. Each dike raise will be approximately five

[28] Light detection and ranging (LIDAR) is a remote sensing technology that measures distance by illuminating a target with a laser and analyzing the reflected light. It is commonly used to make high resolution survey maps.

feet. Dredged material taken from the inside of the disposal area will be used to raise the dike. With each dredging cycle, two feet of freeboard will be modeled from the top of the dike.

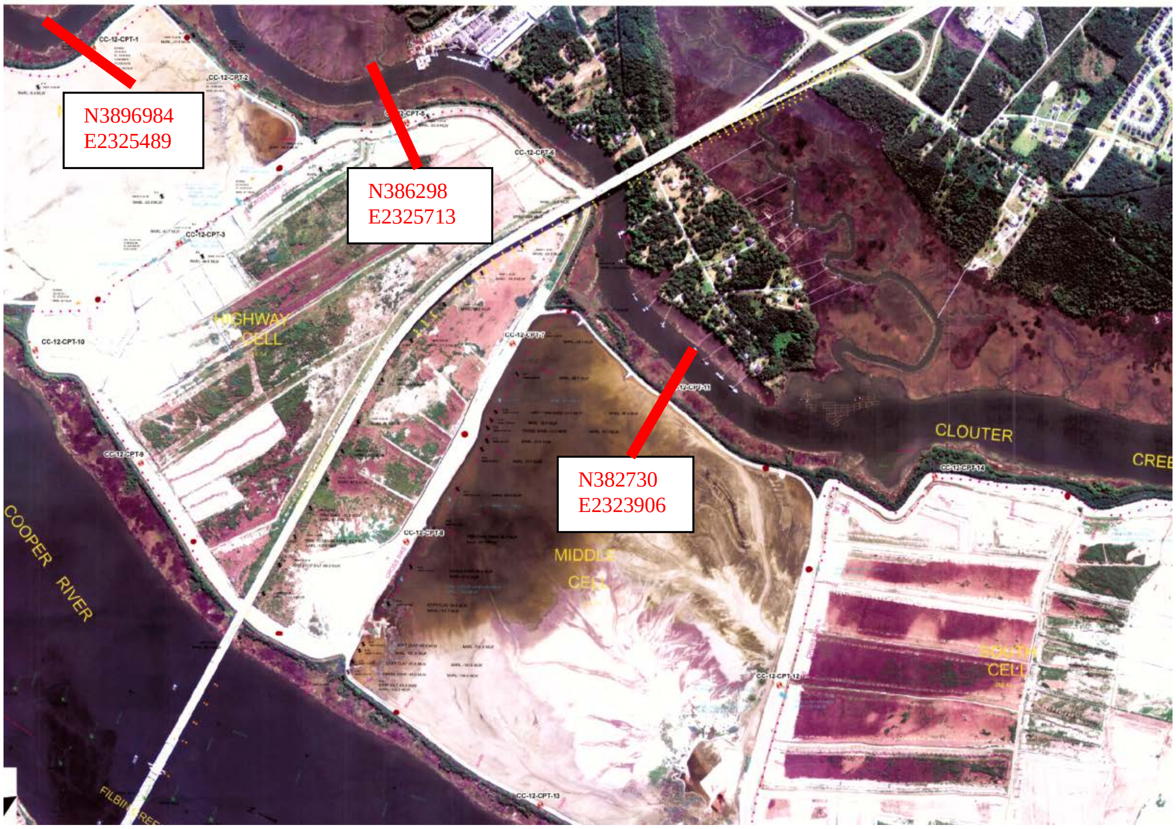


Figure B-73. Modeled cross sections at North Cell, Highway Cell, and Middle Cell

SEEP/W inputs consist of cross sectional geometry, hydraulic conductivity and boundary conditions for the flow domain. Output results from SEEP/W consist of phreatic surface, head distribution, hydraulic gradient, flow directions and flow quantities within the flow domain. Each soil layer was assigned a vertical permeability (k_v) value based on experience with soil types and laboratory permeability tests. The horizontal coefficient of permeability (k_h) of each layer was assumed to be one to two times the vertical permeability. The seepage model follows steady-state conditions, with water surface elevations (headwater) at the crest of the dike.

6.4.1.3 Seepage Analysis Results. As determined by SEEP/W, the seepage pore water pressure within the dike was minor. The phreatic surface exits near the landside toe of the slope with each dredging cycle (2-feet of freeboard). Lateral hydrostatic forces and seepage gradients within the dike and underlying foundation indicate the overall stability of the existing dike is acceptable.

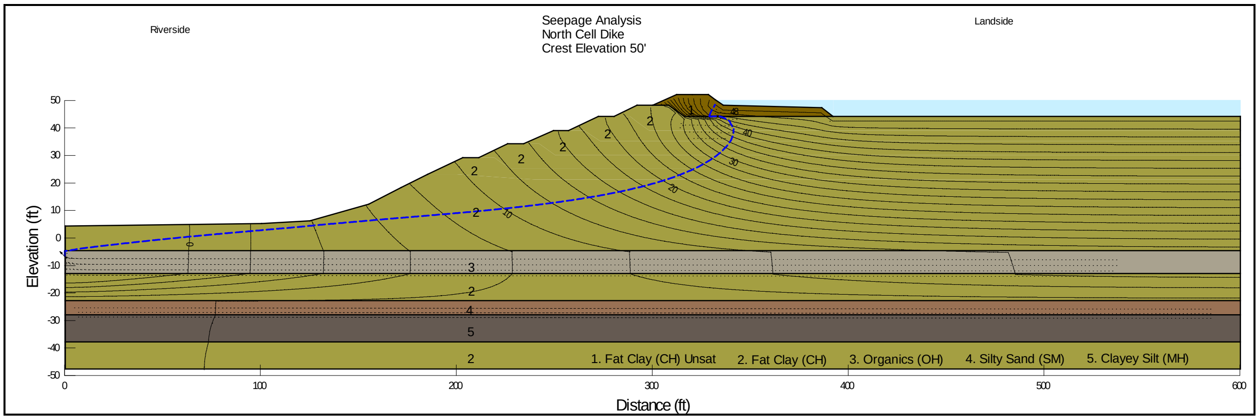


Figure B-74. Seepage analysis of Clouter Creek Disposal Area, North Cell.

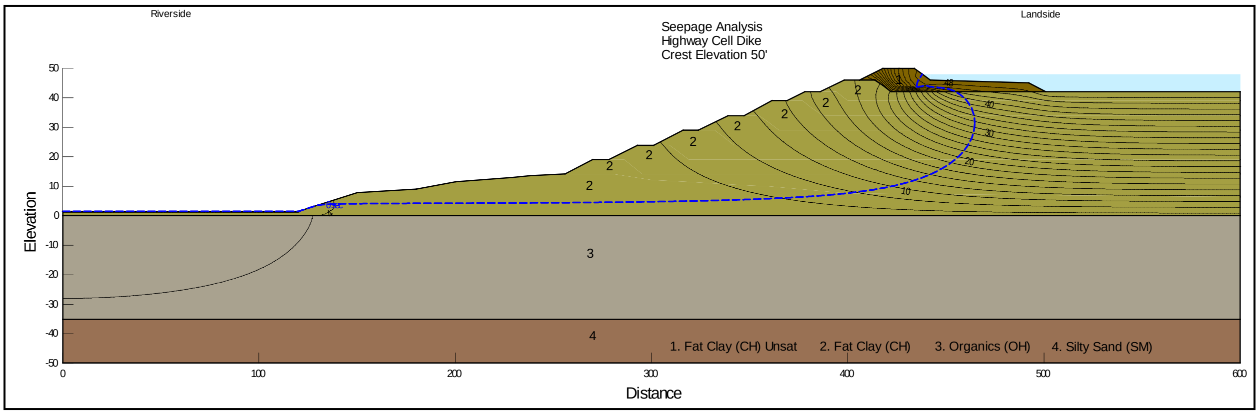


Figure B-75. Seepage analysis of Clouter Creek Disposal Area, Highway Cell

6.4.2 Stability Analysis

6.4.2.1 SLOPE/W. Undrained slope stability analyses were performed using GeoStudio’s SLOPE/W, a two dimensional finite element modeling program. SLOPE/W’s formulation is based on the general limited equilibrium method, and uses an iteration scheme to find the critical slip surface and the corresponding minimum factor of safety. The method of analysis used to determine the factor of safety for Clouter Creek DA is Spencer’s procedure (Spencer 1967, Wright 1970), which is the preferred method of the USACE, per EM 1110-2-1902 Engineering and Design – Slope Stability. Spencer’s procedure fully satisfies static equilibrium for each slice within the failure area. The optimized factors of safety for circular modes of failure were calculated in the analyses. The factors of safety were determined for each dredging cycle after every raise for the fifty year life of the levee. The levee profiles were constructed from the 2013 Clouter Creek Levee cross sections. These cross sections were developed from the 2012 LIDAR topographic survey conducted by the Charleston District (SAC). Soil stratification was determined utilizing data from the 2013 Standard Penetrometer Testing lab results. Soil strength functions were defined, and slip surfaces were specified for the dike raise to elevation 50’. Optimization incrementally alters only portions of the slip surface. After finding the critical slip surface, the new segmental technique is applied to optimize the solution, resulting in a conservatively lower factor of safety than the one obtained for an assumed circular slip surface. The same cross sections were used for both the SLOPE/W analysis and the SEEP/W analysis (North Cell and Highway Cell).

6.4.2.2 Stability Analysis Results.

6.4.2.2.1 North Cell. As determined by SLOPE/W, the factor of safety decreases with each subsequent dike raise to the projected 50-year life cycle elevation of Clouter Creek DA. Utilizing geotextile into the design of the dike increases the factor of safety. As seen by Figures B-76 to B-79, the FS is 0.881 for a dike elevation of 50’ with no geotextile, but increases with each subsequent placement of a geotextile layer. The addition of three geotextile layers at elevations 19’, 28’, and 37’ increases the FS to 1.315, which is above the minimum FS of 1.3 (EM 1110-2-1913) for end of construction.

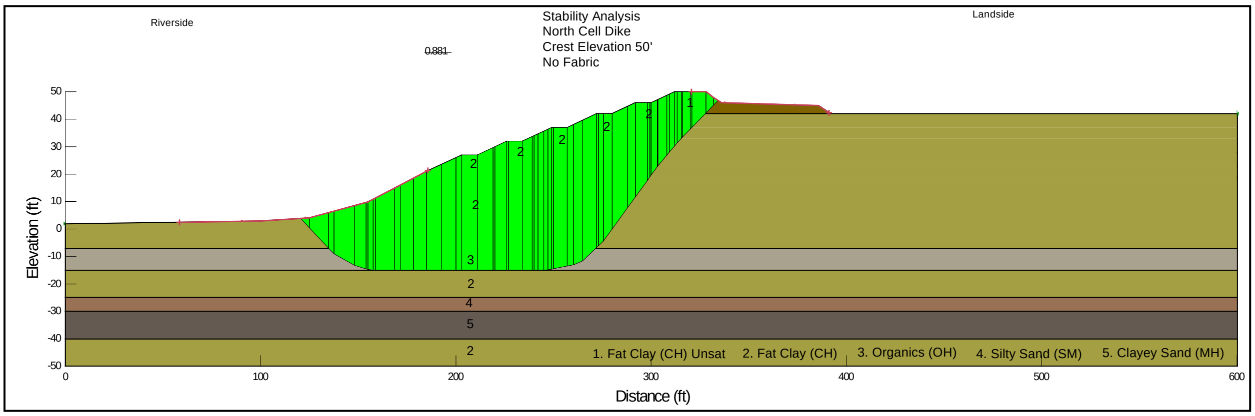


Figure B-76. North Cell. Elevation 50’. No Geotextile. FS = 0.881.

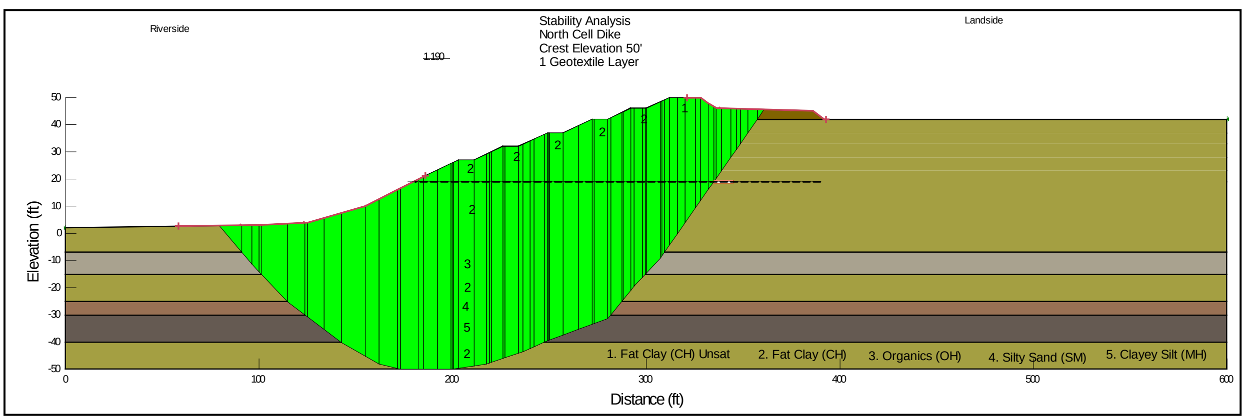


Figure B-77. North Cell. Elevation 50’. 1 Geotextile layer. FS = 1.190.

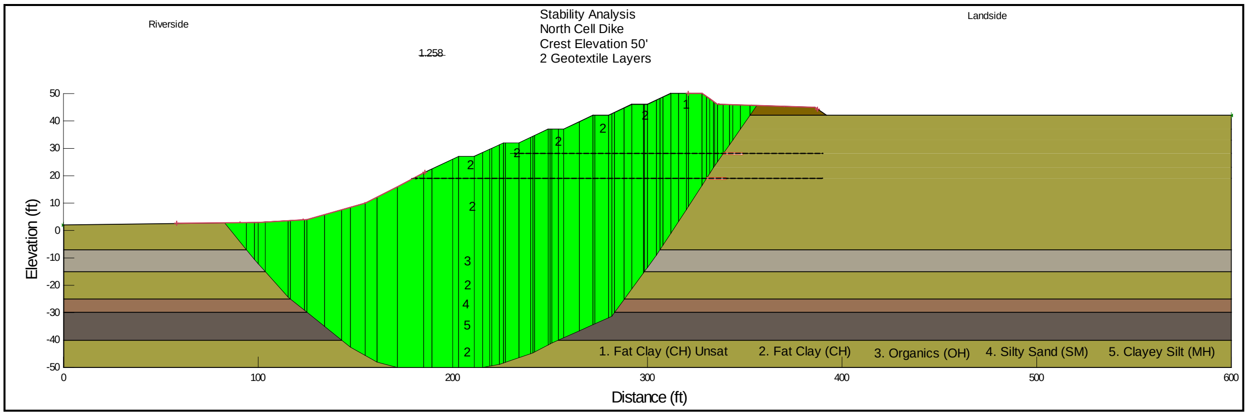


Figure B-78. North Cell. Elevation 50’. 2 Geotextile layers. FS = 1.258.

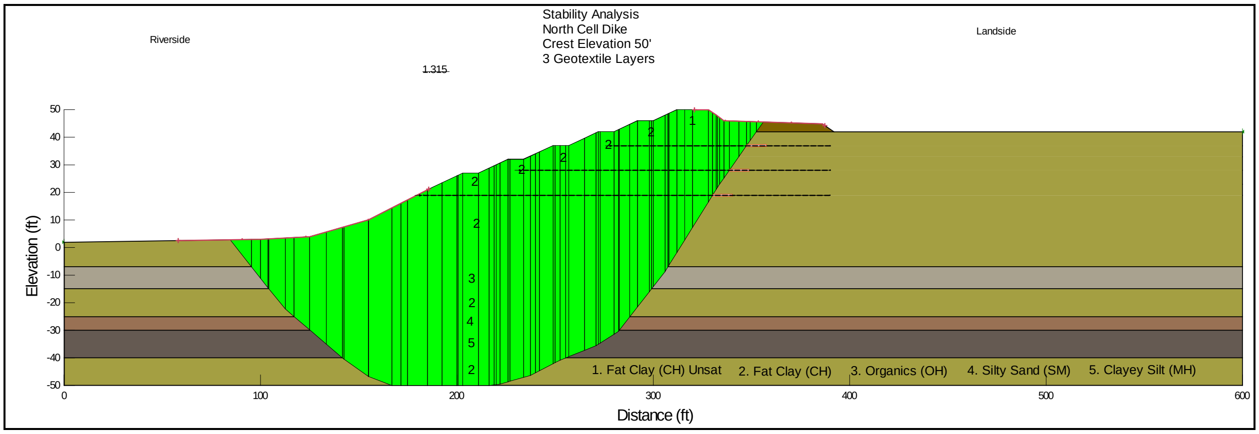


Figure B-79. North Cell. Elevation 50’. 3 Geotextile layers. FS = 1.315

6.4.2.2.2 Highway Cell As determined by SLOPE/W, the factor of safety decreases with each subsequent dike raise to the projected 50-year life cycle elevation of Clouter Creek DA. Utilizing geotextile into the design of the levee increases the factor of safety, however at elevation 50’, the minimum FS is not met with the inclusion of geotextile fabric. The low FS is due to the large (~ 35’) organic layer below the ground surface. Utilizing geotextile, the FS is raised from 0.630 with no geotextile to 1.116 with 5 layers of geotextile (Figures B-80 and B-81).

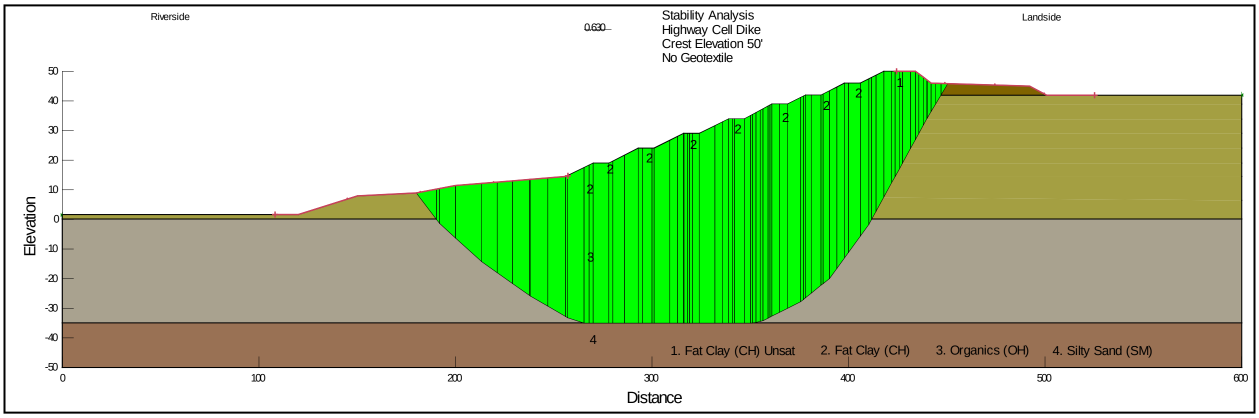


Figure B-80. Highway Cell. Elevation 50’. No Geotextile. FS = 0.630

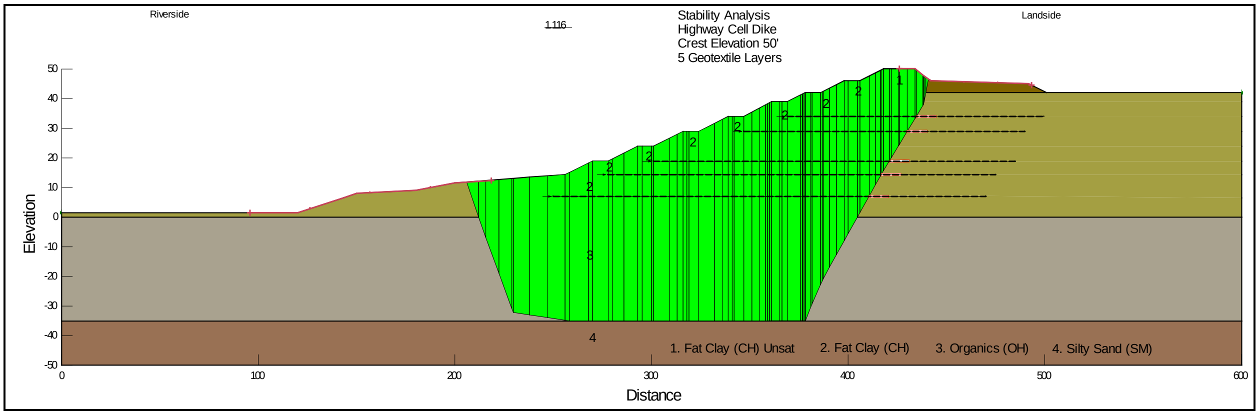


Figure B-81. Highway Cell. Elevation 50'. 5 Geotextile layers. FS = 1.116

Lowering the dike elevation from elevation 50' to elevation 46' increases the FS As seen by Figures B-82 to B-87, the FS is 0.736 for a dike elevation of 46' with no geotextile, but increases with each subsequent placement of a geotextile layer. The addition of five geotextile layers at elevations 7', 14.4', 19', 29, and 34' increases the FS to 1.246, which when rounded, meets the minimum FS of 1.3 (EM 1110-2-1913) for end of construction.

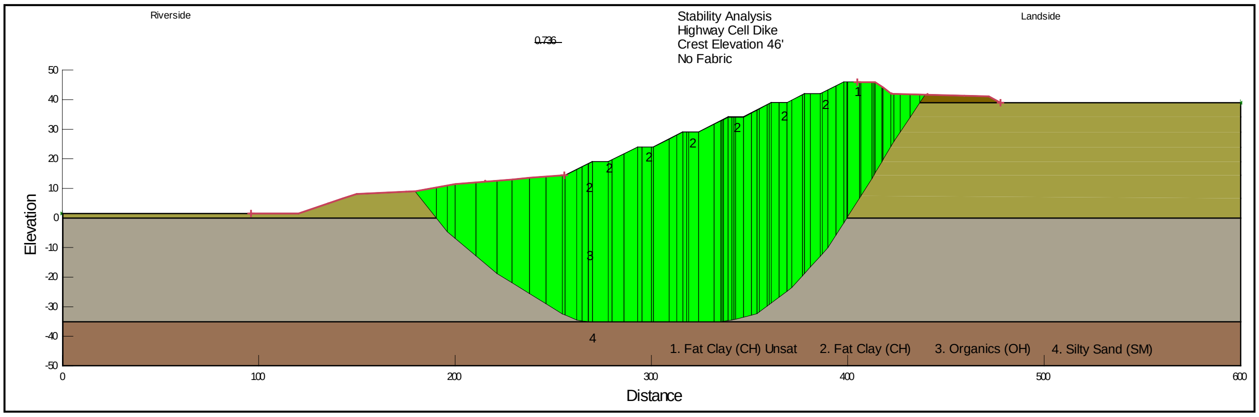


Figure B-82. Highway Cell. Elevation 50'. No Geotextile. FS = 0.736

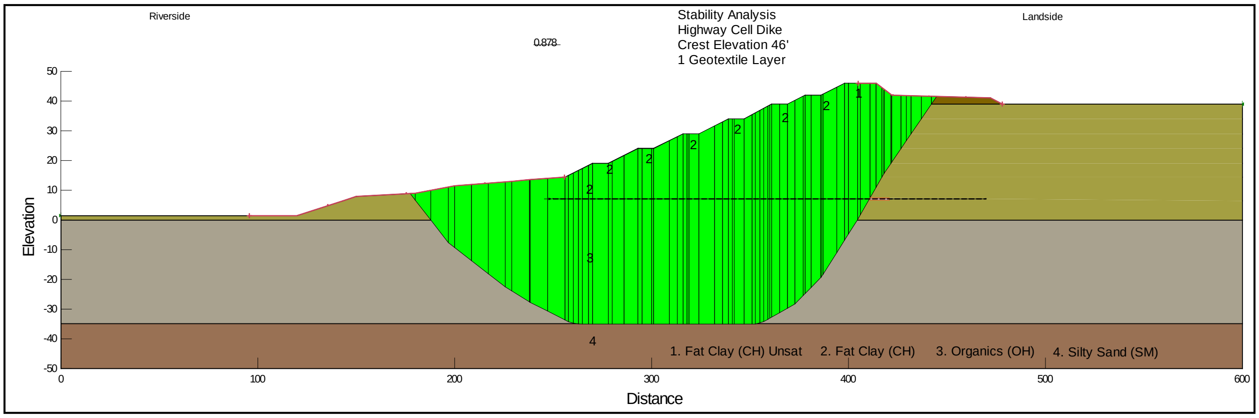


Figure B-83. Highway Cell. Elevation 50'. 1 Geotextile layer. FS = 0.878

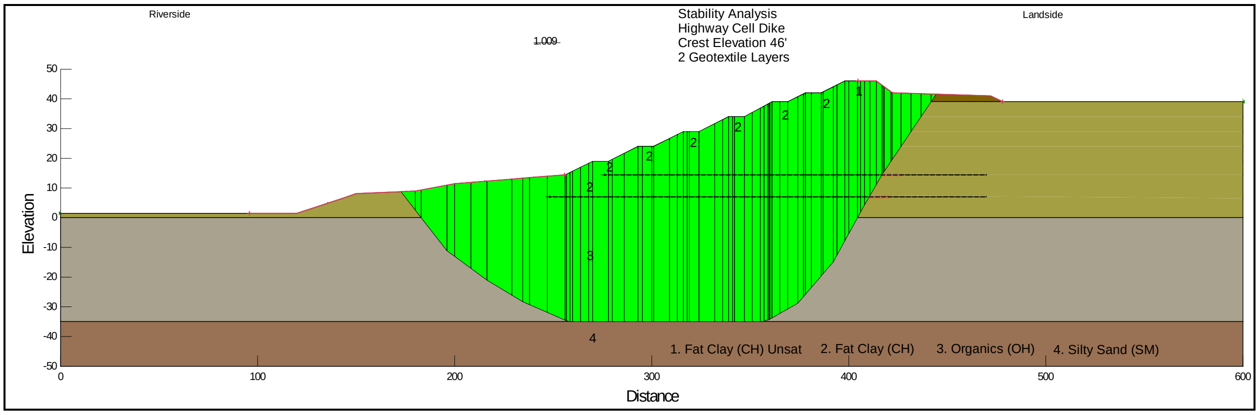


Figure B-84. Highway Cell. Elevation 50’. 2 Geotextile layers. FS = 1.009

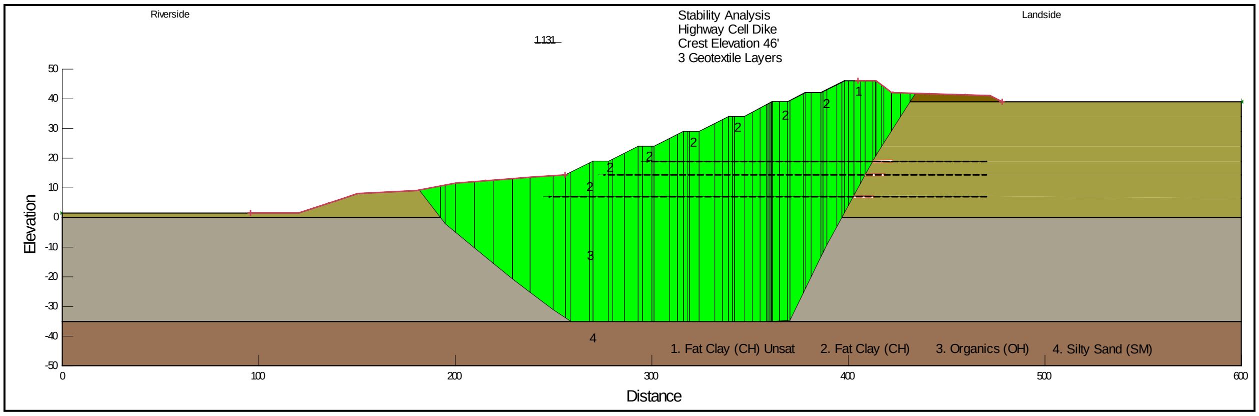


Figure B-85. Highway Cell. Elevation 50’. 3 Geotextile layers. FS = 1.131

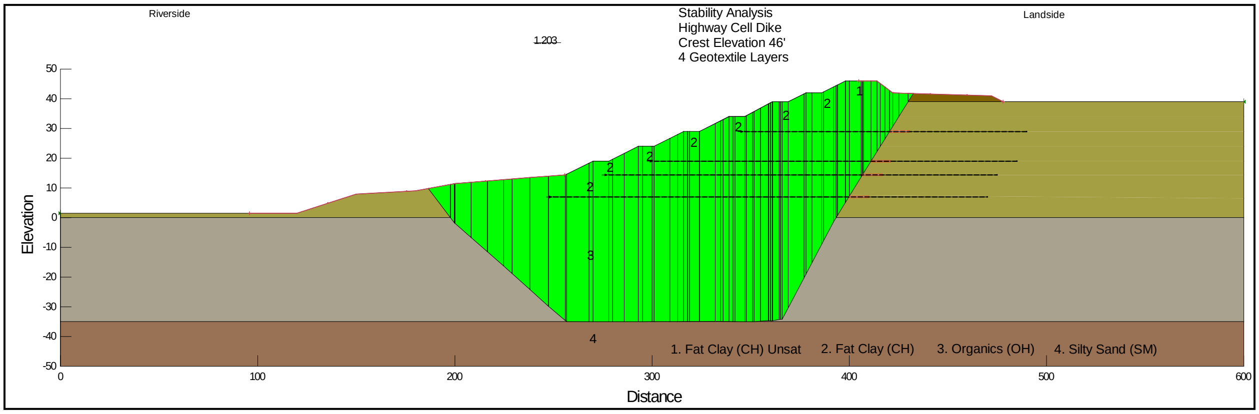


Figure B-86. Highway Cell. Elevation 50’. 4 Geotextile layers. FS = 1.203

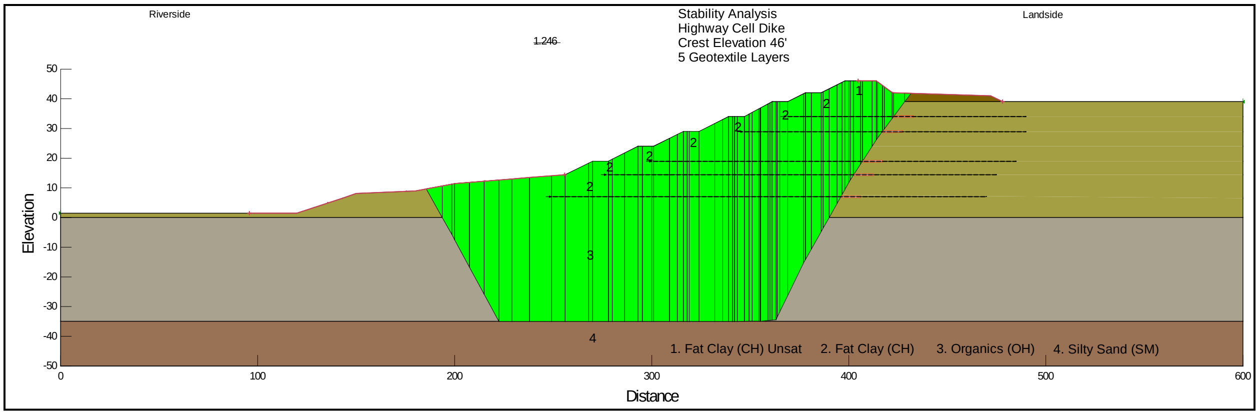


Figure B-87. Highway Cell. Elevation 50’. 5 Geotextile layers. FS = 1.246

6.5 Conclusions and Recommendations

- The 50 year dredged capacity in Clouter Creek Disposal Area has a shortfall of 64 million cubic yards. A raise of 26.9’ is required to accommodate the total 50 year dredged material amount.
- Geotextile fabric is required to raise the dike level.
- The seepage pore water pressure within the dike was minor. The phreatic surface exits near the landside toe of the slope with each dredging cycle (2-feet of freeboard). Lateral hydrostatic forces and seepage gradients within the dike and underlying foundation indicate the overall stability of the existing dike is acceptable.
- For the North Cell, the Factor of Safety is 0.881 for a dike elevation of 50’ with no geotextile, but increases with each subsequent placement of a geotextile layer. The addition of three geotextile layers at elevations 19’, 28’, and 37’ increases the FS to 1.315, which is above the minimum FS of 1.3 (EM 1110-2-1913) for end of construction.
- For the Highway Cell, utilizing geotextile into the design of the levee increases the factor of safety, however at elevation 50’, the minimum FS is not met with the inclusion of geotextile fabric. Utilizing geotextile, the FS is raised from 0.630 with no geotextile to 1.116 with 5 layers of geotextile.
- Lowering the dike elevation from elevation 50’ to elevation 46’ on the Highway Cell, increases the FS, The FS is 0.736 for a dike elevation of 46’ with no geotextile, but increases with the addition of five geotextile layers to 1.246, which when rounded, meets the minimum FS of 1.3 (EM 1110-2-1913) for end of construction.

Although an extensive analysis was performed on the two cross sections in the North Cell and Highway Cell to elevation 50’ NAVD88, further analyses is recommended at numerous cross sections per cell for each dike raise. Current as-builts, as well as refined topographic and subsurface data should be used for each analysis prior to construction. There are an infinite number of section geometries, and only a limited number were analyzed for this investigation.

Foundation improvement is recommended prior to raising the dike to ensure less future settlement and greater stability of the dike. Foundation improvement is one way to increase the foundation soil strengths. Different types of methods include wick drains, sand columns, and stone columns. A seepage, stability, and cost analysis should be performed prior to implementation of any foundation improvements.

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